BASIS OF DESIGN REPORT
PHASE 1A AREA REMEDIATION

LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT

NOVEMBER 1997
Revised March 1998

Prepared by:	Robert E. Carr, P.E. Senior Project Environmental Engineer	3/16/98 Date
Reviewed by:	David L. Bramley, P.E. Project Manager	3/16/98 Date
Project Director:	Christopher R. Klemmer, P.E., LEP Senior Vice President	3/19/98 Date

Fuss & O'Neill Inc. *Consulting Engineers*

146 Hartford Road, Manchester, CT 06040-5921
TEL 860 646-2469 FAX 860 643-6313

1200 Converse Street, Longmeadow, MA 01106-1721
TEL 413 567-9886 FAX 413 567-8936

140 Sherman Street, Fairfield, CT 06430-5822
TEL 203 256-1790 FAX 203 256-5742

1150 Pawtucket Avenue, East Providence, RI 02916-1897
TEL 401 434-9760 FAX 401 434-9920

Fuss & O'Neill Inc. *Consulting Engineers*

146 Hartford Road, Manchester, CT 06040-5921
TEL 860 646-2469 FAX 860 643-6313
INTERNET: www.FandO.com

Other Offices:
Longmeadow, Massachusetts
Fairfield, Connecticut
East Providence, Rhode Island

June 4, 1998

Mr. Martin Beskind
DEP-PERD
Water Management Bureau
79 Elm Street
Hartford, CT 06106-5127

RE: Response to DEP comments dated April 14, 1998 entitled "Comments on Revised 95% Design Package dated March 5/6, 1998 and Responses to Dec. 23, 1997 EPA/DEP Comments"

Dear Mr. Beskind:

Attached are two copies of our responses to the comments of the DEP to Fuss & O'Neill's December 23, 1997 Phase 1A Area 95% Design submittal and our March 5, 1998 *Response to Comments on the 95% Remedial Design*. Two copies have been sent directly to Metcalf & Eddy. As has been the format in the past, the question is included above the answer. When the question is long, an abbreviated version has been entered.

Transmittal Letter dated March 5, 1998

1. *The first bullet indicates that the plans are the 100% Design Plans. As noted in the cover letter from Elise Jakabhazy dated December 23, 1997, the Design Plans and Equipment Specifications do not constitute a 100% design package until all parties have agreed on the latest changes. Upon such agreement, Linemaster must submit a 100% design package for approval by EPA and CTDEP.*

We agree that the Agencies will determine when the 100% Design package is complete.

Response to comments on Nov. 1997 95% Design Package, letter dated March 5, 1998

(A) RESPONSES TO EPA COMMENTS:

2. *Appendices to the Basis of Design Report (Comments to follow as my schedule permits.)*

3. *<u>Air Compressor</u> (Response to EPA Comment 3) The manufacturer's cut sheet indicates that the total CFM capacity of both units is 11.5 cfm at 70 psi discharge pressure. This appears to contradict your response that the dual machine provides 100% excess capacity. Please clarify.*

The comment is correct with the current configuration of the compressors, i.e. they alternate at each cycle. The design maximum capacity of the air compressor, which would potentially occur at system start-up, assumes all pumps are operating at their maximum capacity. The discharge rates will decrease significantly when the pumps have discharged the well casing volume of groundwater from each well. At this point, the groundwater flow will be limited by the aquifer rather than the pumps. The system buffers the potential initial groundwater discharge surge by having the well pump compressed air solenoid valves open on timed delays, which are adjustable from 0.5 to 90 minutes.

Data from groundwater extraction at Linemaster shows total flow rates from all wells between 0.5 and 1.5 gallons per minute (gpm), well below design flow rates. At 1.5 gpm, the required compressor capacity would be approximately 1.2 standard cubic feet per minute (scfm) at 70 psi.

The air compressor system includes two compressor units which currently alternate as the compressor unit cycles on and off. It is possible to configure the system to operate both compressor units together to achieve the total flow capacity of 11.5 cfm, if it is required. In the unlikely event that all of the pumps operated at full capacity, continuous running of the air compressor could also be prevented by reducing the compressed air pressure at the regulators for each line. This would lower pump discharge, and thus compressor air flow requirements.

(B) <u>RESPONSES TO CT DEP COMMENTS</u>:

4. *<u>Air Injection Blower</u>*
(A) Please include the minimum operating printout (at the end of Appendix 4 of the responses) in the Specifications book.

The minimum operating conditions printout will be added to the Specifications.

(B) Once again, we ask what is the minimum anticipated air injection rate? (See our Dec. 23 comment no. 5c.) Despite the impossibility to know the actual minimum injection rate, it is incumbent on the designer to set a

> *minimum flowrate for design purposes (based on injecting air to a selected minimum number of fractures). It is our judgment that air injection rate must always be maintained below air extraction rate at any particular multi-fracture well so as insure vacuum control and avoid buildup of air. Please consider and discuss.*

The blower was selected for operation over a flow range of 280 to 560 scfm at 9 psi. A criteria to be able to supply sufficient air rather than concern over minimum air flows was used to select the specified unit. The design injection rate per fractured well is 40 scfm. This corresponds to two injection fractures, each supplying 20 scfm of make-up air to adjacent vacuum fractures (a given flow in scfm is more easily injected under pressure than extracted under vacuum). Conceivably, the LPA system could be operated with only one well on air injection for a total process flow of 40 scfm. For this case, 110 scfm of bleed air should be discharged from the LPA header bleed valve.

A more likely minimum flow scenario would have two fracture wells on injection to provide access to potential "dead areas" between fracture wells. In this case, the minimum flow is 80 scfm, and the bleed air would be 70 scfm.

> *C. Please address the ability to inject air into lower fractures, considering that the blower discharge pressure was based only on maintaining a safe pressure for injection into the shallow (12 ft. deep) fractures.*

The design air injection pressure of 9 psi at the well head was selected to protect the fractures from over pressure which could cause breakout to the surface or to other fractures. The 9 psi value was based on a ¾ of a psi per foot of depth, applied to the first fracture at 12 feet deep, as recommended by FRx. The actual pressure at the top fracture, or any of the lower fractures, will be reduced by the pressure drop in the ¾ inch PVC piping servicing each fracture. As an example, the pressure loss in this piping at a fracture 35 feet deep would be 0.15 psi, assuming 20 scfm injection air flow. The resulting injection pressure at this fracture would be 8.85 psi.

> *(D) Please advise (1) what the bleed rate must be to permit operation at the anticipated minimum injection rate. (Please express this bleed rate both in absolute flow and as a percent of minimum blower discharge rate), and (2) the anticipated effect on discharge pressure and ability to inject air into the shallow and/or deeper fractures.*

(1) For a minimum injection air requirement for one well (40 scfm), 110 scfm should be discharged as bleed air to meet the blower minimum flow of 150 scfm. This represents 73 percent of minimum blower discharge of 150 scfm. For the two well injection air minimum, 70 scfm of air will be bled from the header, which is 47 percent of total flow.

(2) Injection flow to a given number of fractures will be determined by the supply header pressure and the position of valves in the branch line. At a given pressure in the air injection header and branch lines, the injection air flow rate is determined by the ability of the fractures and formation to convey the air. Bleeding air from the air injection header will not affect the ability to inject air to the minimum number of fractures as long as the design pressure is maintained.

To adjust the bleed valve to maintain the desired injection pressure, the recommended procedure is to:

1. Completely open the header bleed valve and branch line valves for the wells using injection.

2. Set the blower speed to provide the minimum blower flow (150 scfm)

3. Carefully throttle the bleed valve closed until the pressure in the branch lines is 9 psi.

4. Increase the blower speed to attain 9 psi in the branch lines if the bleed valve is completely closed and the pressure is below 9 psi.

The valves on the individual air injection branch lines can be throttled if additional control is required. The air injection system must be re-adjusted each time changes in the air injection configuration are made to ensure the design pressure in the air injection header is maintained.

> *(E) The bleed valve, BV-LP8, is a two inch brass gate valve. Gate valves do not provide good flow control. Please consider whether another type of valve would provide better control.*

While a gate valve may not be the optimum valve type for flow control, as opposed to a butterfly valve, it is a more economical option and is expected to be able to provide sufficient control for its intended use. If control of the air injection system proves to be difficult in practice, the size and type of bleed valve will be re-evaluated.

> *(F) BV-LP8 is not listed in the table at the bottom of the P&ID drawing, nor is it included in Sections 15 and 16 of the Specifications. Please add appropriate information.*

The injection air manifold bleed valve BV-LP8 will be added to Section 16 of the Specifications. It does not appear in the Fracture Well Vault Valve Identification Table on the P&ID drawing because this table describes valves installed in the well vaults only. Valve BV-LP8 is identified in the P&ID line diagram.

5. *Vacuum Blower*
(A) The minimum operating rate has been stated as 50 acfm (24 scfm). This is one twentieth of the maximum operating rate. It is unusual that a blower would be capable of continuous operation over such a wide range of flowrates. Please review this matter with the pump manufacturer.

The vacuum blower minimum flow of 50 acfm was reviewed with the manufacturer and confirmed. Because the blower operates with a secondary air inlet jet which provides cooling, the actual throughput of the blower is approximately 250 scfm at a vacuum flow rate of 50 acfm.

> *(B) Even though the minimum operating rate is stated to be 50 acfm, a minimum operating rate of 140 scfm (300 acfm) is clearly implied by your answer in 6(d) of your response to CTDEP comments. (Operating range of this machine has been given as 300 acfm to 1,000 acfm in the specification sheet.) Please clarify.*

In the design process, a minimum flow performance specification of 300 scfm was used for the vacuum blower. In response to previous CTDEP comments, the minimum flow of the specified blower was determined to be 50 acfm, according to the manufacturer, which satisfies the required minimum flow of the design specifications.

6. *<u>Vapor Blower Heat Exchanger</u> (Response to CTDEP Comment 7) The DEP comment referred to the high pressure side of the exchanger - i.e. the compressed, recycled portion of the blower discharge - not the extracted air/vapor which is still at vacuum. The maximum humidity case should be*

rated to determine if condensation occurs as the atmospheric pressure stream is cooled.

Calculations (attached) show that for design system maximum flow conditions, the heat exchanger service gas stream would be approximately 70 percent humidity at the coolest point of the heat exchanger. If, after evaluating system operating conditions and heat exchanger performance, significant condensation occurs, a condensation drain in the service gas line should be installed.

7. *Portable Air Flow Measurement (CTDEP Comment 8) The flow transmitter shown in the detail on Sheet 14 does not appear in the Specifications book. Please add.*

A specification for the Portable Air Flow Measurement instrument will be added to the Specifications book. The new specification section is attached at the end of this response. In addition, use of the Portable Air Flow Measurement instrument will be discussed in the O&M Manual.

We hope these responses adequately address your comments.

Sincerely,

David L. Bramley

David L. Bramley, PE, LEP
Project Manager

Enclosure

c. w/encl. Gary Kennett - Linemaster
Elise Jakabházy - EPA
Cinthia McLane - M&E

Roots Blower Selection 2.9-97e Monday February 23, 1998 1:34 pm

...r Inj Blower, Min. **Selected Unit**

Model: 68 URAI
Inlet Volume (ACFM): 171 SCFM: 150
Inlet Pressure (PSIA): 14.50
Inlet Temp(Deg.F): 100
Discharge Pressure (PSIA): 23.80
Differential Press. (PSI): 9.30 77%
Ambient Pressure (PSIA): 14.70
Speed (RPM): 855 36%
Brake Horsepower: 14.1
Temperature Rise (Deg. F): 197 82%

Summary: <list>
Gas: AIR
K-Value: 1.395
Specific Gravity: 0.975
Molecular Weight: 28.248
Elevation/Feet: 0
Relative Humidity: 100%
Amb/Jet Temperature: 100
Motor Type: TEFC

Discharge Temperature (Deg. F): 297
Discharge Volume (ACFM): 140
Gear Tip Speed (FPM): 1344
Estimated B10 Bearing Life (HRS): 153000
Estimated Noise Level at 1 Meter (DBA): 80.2

<Esc> Print Proposal, <F7> Cancel, <F10> Next, <F8> Previous

Roots Blower Selection 2.9-97e Monday February 23, 1998 1:35 pm

Air Inj. Blower, Max. **Selected Unit**

Model: 68 URAI
Inlet Volume (ACFM): 712 SCFM: 625
Inlet Pressure (PSIA): 14.50
Inlet Temp(Deg.F): 100
Discharge Pressure (PSIA): 23.80
Differential Press. (PSI): 9.30 77%
Ambient Pressure (PSIA): 14.70
Speed (RPM): 2226 94%
Brake Horsepower: 38.6
Temperature Rise (Deg. F): 129 53%

Summary: <list>
Gas: AIR
K-Value: 1.395
Specific Gravity: 0.975
Molecular Weight: 28.248
Elevation/Feet: 0
Relative Humidity: 100%
Amb/Jet Temperature: 100
Motor Type: TEFC

Discharge Temperature (Deg. F): 229
Discharge Volume (ACFM): 534
Gear Tip Speed (FPM): 3500
Estimated B10 Bearing Life (HRS): 59000
Estimated Noise Level at 1 Meter (DBA): 94.7

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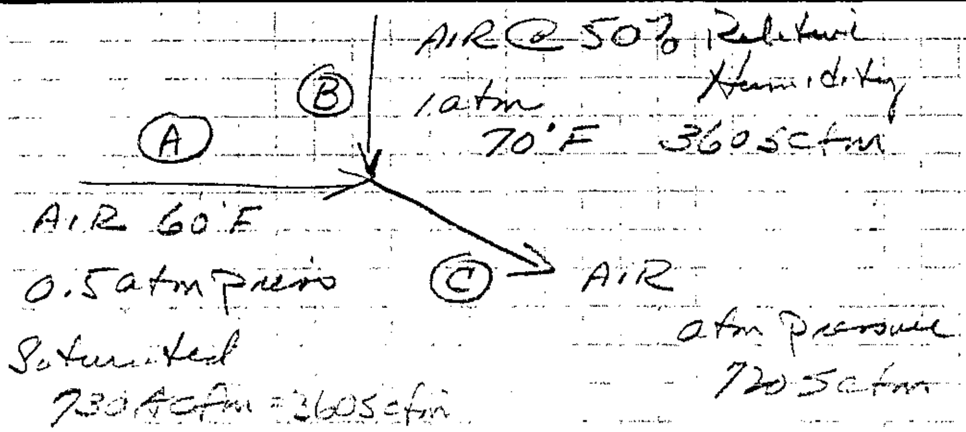
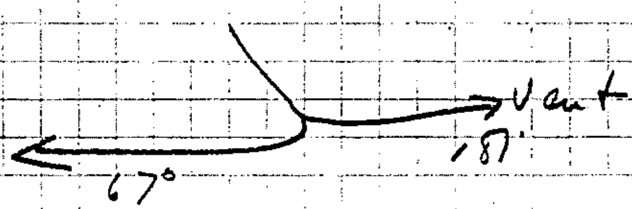
HUMIDITY OF AIR RECYCLE STREAM AFTER HEAT EX.





Stream (A) FROM Perry Pg 12-7+

$11.08 \times 10^{-3} \frac{gm\,H_2O}{lb\,dry\,air} \times \frac{lb\,dry\,air}{13.329\,ft^3 (std)} = 0.831 \times 10^{-3}$

∴ at atm pressure

$11.08 \times 10^{-3} \frac{gm\,H_2O}{lb\,dry\,air} \times \frac{lb\,dry\,air}{13.329\,std\,ft^3} \times \frac{1\,std\,ft^3}{0.5\,Act\,ft^3}$

Stream (B) $1.582 \times 10^{-2} \frac{gm\,H_2O}{lb\,dry\,air} \times \frac{lb\,dry\,air}{13.687\,std\,ft^3} (\times 0.5\,R.H.) =$

$0.578 \times 10^{-3}\ gm/std\,ft^3$

Since Stream (C) is a 1/1 mix, we have

$\frac{(0.831 + .578) \times 10^{-3}\ gm\,H_2O}{2\,ft^3} = 0.7045 \times 10^{-3} \frac{gm\,H_2O}{ft^3}$

Fuss & O'Neill Inc. *Consulting Engineers*

PREPARED BY	DATE	CHECKED BY	DATE	PROJECT NO
HEK	6-3-98	TCS	6-3-98	86088 A5

@ 67°F, Saturation $\frac{gm}{ft^3}$ = $14.25 \times 10^{-3} \frac{gm}{lb\,dry\,air} \times \frac{lb\,dry\,air}{13.57\,ft^3}$

$= 1.05 \times 10^{-3}\ gm/ft^3$

Relative Humidity at Heat Exchanger Airmec Discharge

is $\dfrac{0.7045 \times 10^{-3}\ gm\ H_2O/ft^3}{1.05 \times 10^{-3}\ gm\ H_2O/ft^3}$

$= 67\%$

PORTABLE AIR FLOW RATE INDICATOR

Item:	Soil Vapor and Injection Air Portable Flow Rate Indicator
Quantity:	1
Pipe:	2 inch PVC Schedule 40, 65 inch length
Normal Flow:	12 acfm @ -5.9 psig (vacuum) 13 acfm @ 9 psig (low pressure air)
Flow Range:	0 to 20 acfm at either -5.9 or +9 psig
Parts:	PVC piping, orifice plate and holder, differential pressure gauge, pressure indicator, interconnective tubing, fittings and vent valves
Gage Connections:	1/4 inch NPT extension nipples, vent valves, 3/16 ID barbed male adapters, and rubber tubing sufficient to reach gages.
Installation:	As shown on Sheet 14 of design drawings. Carrier ring and orifice plate (one piece construction) mounted between standard ANSI 125#/150# PVC flanges. Units will be self-centering between the flanges. Differential pressure gage will be attached to orifice plate per manufacturers instructions. Pressure indicator will be attached to pressure barbed fitting on "high" side for LPA flow, and on "low" side for soil vapor (vacuum) flow.
Construction:	Orifice Plate: Unitized stainless steel orifice plate and carrier ring, Manville 961 Kevlar integral gaskets, concentric bore. Brass or stainless steel fittings, rubber tubing, die cast aluminum gages, PVC valves.
Manufacturer:	Lambda Square, Inc. (Oripac); Magnehelic (Gages) Hose Connectors: Dixon Valve And Coupling Co. Flexible Hose: New Age Industries, supplied by Faxon Engineering
Model Number:	Oripac: 5300-02-1.3573-Sch 40 Magnehelic: Model 2210 Hose Connectors: 1" male NPT plugs and couplings, straight through quick connect fittings Flexible Hose: 1" NEWFLEX VFH

Representative:

Lambda Square, Inc.
P.O. Box 1119-M
Bay Shore, N.Y. 11706
Attn: Rob Lang, Jr.
Phone: (516) 587-1000
Fax: (516) 587-1011

Faxon Engineering
467 New Park Ave.
West Hartford, CT 06110
Phone: (860) 236-4266

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
REGION 1
JOHN F. KENNEDY FEDERAL BUILDING
BOSTON, MASSACHUSETTS 02203-0001

SENT VIA FACSIMILE AND FIRST CLASS MAIL

April 23, 1998

METCALF & EDDY, INC.
APR 27
RECEIVED

Mr. Gary Kennett
Project Coordinator
Linemaster Switch Corporation
29 Plaine Hill Road / P.O. Box 238
Woodstock, CT 06281

Re: **Linemaster Switch Corporation Superfund Site: Response to Comments on the Remedial System Monitoring Plan; and, Comments on the Dewatering System Start-up Plan.**

Dear Mr. Kennett:

The U.S. Environmental Protection Agency (EPA) has reviewed the Response to Comments on the Remedial System Monitoring Plan and the Dewatering System Start-up Plan prepared Fuss & O'Neill on behalf of Linemaster Switch Corporation dated March 1998. The Responses, the Plans and Specifications were reviewed in accordance with the Consent Decree issued in United States of America and The State of Connecticut v. Linemaster Switch Corporation, Inc., D. Conn. 1995, Civil Action Nos. 3:94CV01709, 3:94CV01710, (the "Consent Decree") for Remedial Design and Remedial Action, entered on January 4, 1995.

Enclosed, please find two sets of EPA comments for your review: 1) for the Response to Comments for the Remedial System Monitoring Plan; and, 2) for the Comments on the Dewatering System Start-up Plan.

Additionally, please review comments previously sent to you dated April 14, 1998 from Mike Beskind of the Connecticut Department of Environmental Protection (CTDEP) addressed to EPA. EPA would particularly like to note that since your Phase 1A remedial system was installed *prior* to receiving all final comments from EPA and CTDEP, we are left with few options. While we still have some concerns regarding the air injection blower, the vacuum blower and the vapor blower heat exchanger, we recognize that since the system is already installed – the design engineers (F&O) should have their chance to prove that their system truly meets their design expectations. Please note, however, that if during operation our concerns persist regarding any of the EPA and/or the CTDEP's past comments (specifically those dated December 9, 1997 and CTDEP's comments dated April 14, 1998), we may require Linemaster to replace any equipment not meeting performance expectations.

c: C. McLane
W. Dies
J. Ford
File WA #12

Should you have any questions regarding these two sets of comments, please do not hesitate to call me at (617) 573-5760.

Sincerely,

Elise Jakabházy,
EPA Project Manager

2 encl.

c. Cinthia McLane, M&E
Martin Beskind, CTDEP
David Bramley, F&O

EPA Review Comments
Dewatering System Start-up Plan
Phase 1A Area Remediation
Linemaster Switch Corporation
Prepared by Fuss & O'Neill
March 1998

<u>No.</u>	<u>Item & Comment Description:</u>
1.	**Section 3.1 (6) <u>Compressed Air System</u>, p.2.** Please include any appropriate safety precautions to be taken by the person handling the water. Will there be a Health and Safety Plan associated with system start-up/operation?
2.	**Section 3.1 (10) <u>Compressed Air System</u>, p. 3.** Please clarify which ball valve is to be closed.

EPA Review Comments
Response to Comments on the Remedial System Monitoring Plan and
Revised Remedial System Monitoring Plan
Linemaster Switch Corporation
Prepared by Fuss & O'Neill
March 1998

No. **Item & Comment Description:**

1. **General.** Fuss & O'Neill's Response to Comments generally provided a lot of detail that, in some cases, was not incorporated into the revised Monitoring Plan. Although it is not necessary or appropriate to incorporate all of the information provided in the responses into the plan, some of the information would make the Plan a more useful, stand-alone document. Specifically, detail provided in the responses to EPA comment 8 (page 5 of Response to Comment, bottom paragraph) and EPA comment 14 must be included in the Monitoring Plan. The response to Comment 8 gives a useful description of how data generated by TDR measurements and water level measurements will be used to better assess subsurface conditions. The response to comment 14 provides a description of anticipated modifications to the groundwater sampling program as dewatering proceeds.

2. **Response to Comment 4.** We concur that the benefit of operating at a high flow rate once remediation has been determined to be diffusion-limited may result in only a slightly higher rate of mass removal, and did not intend to suggest continuous operation at the same (high) flow rate in the comment. Rather, as discussed at the November 7, 1997 meeting, mass removal will be accomplished more expediently by continuous operation at a low-velocity, reduced flow rate than by intermittent operation, due to the concentration gradient produced during continuous operation. Please re-evaluate the use of low-flow operation as an alternative to intermittent operation. It is unclear from the response if this is one of the three modes of operation being considered for evaluation for long-term operation.

3. **Response to Comment 14.** The dewatering portion of the system is now activated. Were the groundwater samples from the piezometers collected prior to startup? Will there be some down time again in the future? If so the part of the *Sampling, Analysis, and Monitoring Plan - Phase 1A Area Remedial System* that covers this sampling effort must be submitted to the agencies as soon as possible. If the samples will be collected after system startup, please indicate the schedule for the sampling and for the submittal of that part of the *Plan*.

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paper

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LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT

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PHASE 1A AREA REMEDIATION
LINEMASTER SWITCH CORPORATION
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1.0 INTRODUCTION

1.1 General

The development of the design criteria and this basis of design report have been an on-going, cooperative effort since approximately January 1995. The data developed from the 1994 Dual Vacuum Extraction (DVE) pilot test in November 1994 indicated that there was insufficient data on soil characteristics to develop a comprehensive Conceptual Remedial Design. It also indicated that enhancements to the natural characteristics of the overburden would be required to achieve adequate air and groundwater flow. Evaluation of existing enhancement and alternative remedial techniques indicated that soil fracturing would result in increased fluid delivery and recovery rates. In June 1995, we prepared an RFP for soil fracturing and distributed it to potential hydraulic and pneumatic soil fracturing contractors. In July we selected FRx to perform hydraulic fracturing after discussion during a conference call with the Agencies on July 20, 1995. The fracturing pilot test was conducted in November and December 1995 in accordance with the September 1995 Work Plan.

Concurrent with the evaluation of soil remediation/enhancement options, we undertook a boring program for two purposes. The first was in support of an expansion of the production facility to the north. The second was to more clearly define the Zone 1 area, the area within the 100 microgram per kilogram (ug/kg) soil trichloroethene (TCE) isopleth. In February 1996 we presented our *Zone 1 Delineation Report* which described the area impacted by TCE.

In addition to analyzing soil samples for VOC concentrations, analyses were conducted on selected samples to determine the physical characteristics needed to develop design criteria for the remedial system. These analyses were discussed in our Technical Memorandum entitled *Overburden Geology and Physical Characteristics* dated February 1996, a copy of which is included in Appendix A.

The Record of Decision (ROD) requires remediation of the soil to a trichloroethene (TCE) concentration of 5 ug/kg. While this concentration is still the goal of the remedial system, we performed a feasibility analysis using a target soil concentration of 100 ug/kg, consistent with the Connecticut Remediation Standard Regulations (Appendix B). On March 6, 1996, we presented to the Agency the results of our data analysis. This analysis predicted that remediation of the Zone 1 area to the target cleanup concentration (100 ug/kg) in a responsive time frame likely would be technically infeasible. In response, the Agency requested a proposal for a modification to the conceptual remedial design for the Zone 1 Area which we provided on March 11 (Appendix C).

On May 2, 1996, a conference call was held to discuss a revised conceptual design. During this call the Agency reiterated that the goal was to achieve maximum mass removal of VOCs in the newly defined Phase 1A Area. At a meeting of all parties on May 16 the Phase 1A Area was delineated as the area within the TCE 1,000 ug/kg soil concentration isopleth (Appendix D and E). All agreed that the Phase 1A Area would be a target area to determine the effectiveness of a remedial system incorporating hydraulically fractured overburden wells, overburden dewatering and soil vapor extraction. The EPA position was explained in a letter dated May 29, 1996 (Appendix F). On July 29, 1996, the Agency concurred with the locations selected for the fractured recovery wells. The performance of these wells was expected to provide an indication, via the success of VOC mass removal, of the likelihood of successfully remediating the Zone 1 Area. Based on subsequent correspondence and discussions held during telephone conference calls, the design criteria and remedial design strategies have evolved to what is presented herein.

1.2 Purpose

The purpose of this Basis of Design Report is to present the criteria used to size and select the components of the Phase 1A Area remedial system. It also summarizes the characteristics of the major system components. Dewatering/soil vapor extraction (DVE) is the selected remediation technology as discussed in the *Record of Decision* (EPA, July 1993).

1.3 Site Description

Linemaster Switch Corporation is located on Plaine Hill Road in Woodstock, Connecticut. The company manufactures foot switches at a facility, situated on a topographic high point, located in the central portion of the 45-acre property. The area delineated as the Phase 1A Area is located east of the manufacturing facility, and is depicted in Figure 1.

1.4 Previous Investigations

A number of investigations have been performed at the site from 1988 to 1996. The following Fuss & O'Neill reports were used to develop remediation design conditions within the Phase 1A area:

- *Remedial Investigation/Feasibility Study - Linemaster Switch Corporation, Woodstock, Connecticut, Latest Revision December 1992*
- *Fracture Well Modeling Status/Methodology - Remediation of Zone 1 Linemaster Switch Corporation, Woodstock, Connecticut, November, 1995*
- *Soil Fracturing Pilot Test Results Text, Table and Figures - Linemaster Switch Corporation, Woodstock, Connecticut, January, 1996*
- *Zone 1 Delineation Report - Linemaster Switch Corporation, Woodstock, Connecticut, February, 1996*
- *Overburden Geology and Physical Characteristics Technical Memorandum - Linemaster Switch Corporation, Woodstock, Connecticut, February, 1996*
- *Work Plan for Hydraulic Fracturing of Phase 1A Area Wells - Linemaster Switch Corporation, Woodstock, Connecticut, September 1995*
- *Phase 1A Area Fracturing Report - Linemaster Switch Corporation, Woodstock, Connecticut, Revised March 1998*

2.0 SITE INVESTIGATION SUMMARY

2.1 Site Geology

Overburden deposits at the Linemaster facility have been mapped as glacial till (Randall et al., 1996). During drilling operations at the site from 1988 through 1997, till deposits have been encountered at all boring locations. The glacial till consists of dense, compact, non-sorted and non-stratified mixture of clay, silt, sand, gravel, cobbles, boulders and angular rock fragments. The distribution of fine and coarse grained materials varies laterally and vertically throughout the site.

Drilling investigations within the Phase 1A Area indicate two glacial till units: a dense, brownish, upper till, and a dense, grayish, lower till. Field observations indicate that these two units appear compositionally similar, however, the lower till appears to be slightly finer grained and contains less sand than the upper till. The transition area between the upper and lower till units within the Phase 1A Area is between 15 to 18 feet below grade.

Bedrock at the site is mapped as Hebron Gneiss, which consists of interlayered dark-gray schist and greenish-gray fine to medium grained gneiss (Rodgers, 1985). Depth to bedrock at the site ranges from 30 feet below grade southeast of the manufacturing building to 60-70 below grade northeast of the building.

2.2 Site Hydrogeology

The quality of groundwater in the area of the Linemaster facility is classified by the Connecticut Department of Environmental Protection as GA (CT DEP, 1993). Groundwater classified as GA is defined by CT DEP as groundwater within the area of existing private water-supply wells or an area with the potential to provide water to public or private water-supply wells. The CT DEP presumes that groundwater in such an area is, at a minimum, suitable for drinking or other domestic uses without treatment. The designated uses for Class GA groundwater are as existing private and potential public or private supplies of water suitable for drinking without treatment and as baseflow for hydraulically-connected surface water bodies (CT DEP, 1996).

The overburden and bedrock aquifers are used in the site vicinity to supply potable water for nearby residences and businesses. Groundwater contaminant distribution data and previous investigations indicate that these two aquifers are hydraulically connected.

Groundwater flow within the overburden aquifer generally parallels the ground surface topography. As indicated in Figure 2, a November 4, 1996 overburden aquifer groundwater contour map, groundwater flows radially outward from the Linemaster facility location.

Average linear groundwater flow velocities range from 0.0004 feet per day (ft/day) to 0.10 ft/day east of the manufacturing building, and from 0.07 ft/day to 0.24 ft/day further down the hill to the east and adjacent to Route 169 (F&O, 1992).

2.2.1 Hydraulic Conductivity

Hydraulic conductivity is a measure of the ability of a porous medium to transmit water, and is described in units of velocity. The hydraulic conductivity of the till is best characterized by slug test data analyses. The horizontal hydraulic conductivity of the overburden aquifer is estimated to range from approximately 0.015 to 0.003 feet per day (5.3×10^{-6} to 1.1×10^{-6} centimeters per second (cm/s)). The greater value is representative of the top 15 to 20 feet of till and the lower value is representative of the overburden deposits below a depth of 20 feet.

2.3 Summary of Soil Sampling Analytical Results

Analytical results of soil samples collected prior to July 1991 were summarized in the *Remedial Investigation/Feasibility Study* (RI/FS) (F&O, 1992). Following completion of the RI/FS, investigations were conducted to determine the degree and extent of VOC contaminated soils in the vicinity of the former dry well located immediately east of the manufacturing facility. Analytical results of soil samples collected after July 1991 and before December 1995 are summarized in the *Zone 1 Delineation Report* (F&O, 1996).

The objective of these subsequent investigations was to establish a study area where soil TCE concentrations exceeded the target concentration. This area was determined by identifying locations where TCE, the most predominant VOC, exceeded the CT DEP pollutant mobility criteria. For GA classified areas, the pollutant mobility criteria for TCE is 100 ug/kg. The resulting study area, now referred to as Zone 1, is defined as the region in which TCE concentrations exceed 100 ug/kg. Although other VOCs exceed CT DEP soil clean-up criteria, the region in which they do so is encompassed by the Zone 1 area.

3.0 REMEDIATION SELECTION

The *Record of Decision* states that Linemaster will dewater the overburden and use soil vapor extraction (SVE) technology to remediate soils. Technical negotiations and discussions with EPA and DEP have resulted in a design intended to address contaminated soils in an area surrounding the former dry well that has been designated as the Phase 1A Area. The Phase 1A Area encompasses an area where soil TCE concentrations exceed 1,000 ug/kg. This area is located east of the manufacturing facility and is depicted in Figure 1. To assess the feasibility of dewatering and using SVE, Fuss & O'Neill, Inc. evaluated the physical characteristics of the overburden deposits. These characteristics, including hydraulic conductivity and air permeability, were summarized in the *Overburden Geology and Physical Characteristics Technical Memorandum*, dated February, 1996.

3.1 Soil Characteristics Enhancement

According to EPA soil vapor extraction reference guidelines (February, 1991), VOC removal has been documented for soils with hydraulic conductivities ranging from 1×10^{-3} to 1×10^{-6} cm/s. The hydraulic conductivity values for soils at the Linemaster site were determined to be at or below the lower documented SVE success range. A dual phase vacuum extraction pilot test, performed in December 1994, indicated that it would be necessary to enhance the permeability and hydraulic conductivity of the overburden to achieve greater air and groundwater flow and expand the area of influence of an extraction well. After an evaluation of options, hydraulic soil fracturing was selected as a remedial approach to improve subsurface fluid delivery and recovery. Subsequent to hydraulic fracturing, a second pilot test was conducted during November and December 1995 to assess the effect of fracturing on dewatering and soil vapor extraction.

Hydraulic soil fracturing was conducted to extend a series of fractures radially from each extraction well. The propagated fractures were filled with coarse sand to increase longevity. These sand-filled fractures have been documented to effect an increase in fluid recovery and delivery rates. Correspondingly, this translates into an increase in VOC recovery rates and, therefore, a reduction in remedial duration. The fracturing process and 1995 pilot test are summarized in <u>Section 4.0</u>.

4.0 HYDRAULIC FRACTURING PILOT TEST

The remedial design incorporates eleven hydraulically fractured dewatering/SVE wells, and the existing shallow bedrock monitoring well MW-10sb, immediately east of the Linemaster manufacturing building. The feasibility of soil fracturing at the site was evaluated from the November-December 1995 soil fracturing pilot test results.

The hydraulic fracturing pilot test was performed in two phases. In the first phase, the feasibility of creating hydraulic fractures at various depths in the overburden was evaluated. In the second phase of the test, the ability to dewater the area in the vicinity of the fractured wells and the performance of vacuum extraction from the same wells were assessed.

Two fractured well borings, FW-A and FW-B, were drilled to the bedrock surface using hollow-stem augers. The wells were constructed of six-inch diameter PVC casing with a solid bottom grouted in place at the base of the borehole. Fractures were initiated in each well from slots cut in the PVC casing and surrounding grout using a high pressure (10,000 psi) water jet cutting head lowered to the desired depth. Once a cut was made, a straddle packer was lowered to isolate the notch, and a guar gum gel with well sorted sand was injected into the notch. The gel/sand mix was injected at a sufficient pressure to propagate a fracture through the overburden formation. After injection, the guar gum gel decomposes and is removed leaving a thin, elliptical, sand-filled fracture at the designated depth.

Three fractures were created in FW-A at 8, 18, and 28 feet below grade. Seven fractures were created in FW-B at 8, 13, 18, 23, 28, 33, and 38 feet below grade. Borings were drilled at specific distances from each fractured well to determine the location and estimate the configuration of each fracture. The results of the fracture confirmation borings indicate that the fractures likely climbed as they propagated away from the fracture well, especially with depth.

During the 1995 pilot test, groundwater in each fracture well was pumped to lower the level in the well to dewater the fractures. The long term flow capacity of well FW-A was 0.05 gallons per minute (gpm), and 0.2 gpm in well FW-B. The soil fracture pilot test results are detailed in the *Soil Fracturing Pilot Test Results* report (F&O, January 1996).

5.0 AIR MODELING

The results of the hydraulic fracturing pilot test indicated air permeability rates ranging from 4.1×10^{-8} to 5.8×10^{-10} cm^2 in the upper till, and 1.6×10^{-10} cm^2 in the lower till. A one dimensional model initially was used to calculate fracture well air flow rates. For till fractures above 12 feet, an average air permeability of 2.9×10^{-9} cm^2 was used, and for till fractures below 12 feet, a conservative air permeability of 1.5×10^{-9} cm^2 was used in the model. This model predicted a cumulative flow rate of 36 scfm for two fractures with the top fracture on vacuum and the bottom on an injection (Appendix G).

Subsequently, the modeling effort was refined using a three dimensional model using a constant air permeability of 2.9×10^{-9} cm^2 throughout the till stratum. This higher value in the lower till would result in a conservatively higher estimate of the air flow rate when all the fractures are under vacuum. The air permeability of the sand was assumed to be 1,000 times that of the soil (2.9×10^{-6} cm^2) and the fracture thickness was taken to be 0.02 feet. Based on this three dimensional model, significant pressure drops are predicted in the fractures that will reduce the air flow rate. Using an average of 3.4 fractures per well, the predicted air flow rate is 21.9 scfm. Because the air permeability of the lower till will be less than 2.9×10^{-9} cm^2, the actual flow should be less than the modeled flow rate. Therefore, the design flow rate of 40 scfm per well includes a safety factor of two.

6.0 FORM AND DISTRIBUTION OF HYDRAULIC FRACTURES

Evaluation of the geometry and distribution of the hydraulic fractures propagated at the Linemaster site requires consideration of fracturing records (principally uplift data), hydraulic and pneumatic interaction testing results, and confirmation boring findings. The integrated interpretation of these data defines a complex system of horizontal, sub-horizontal, and steeply dipping fractures.

Uplift data indicate that many of the fractures were circular in shape, with diameters of 20 to 40 feet. The rest were elliptical, with major diameters from 12 to 50 feet and minor diameters from 7 to 38 feet. Most of the fractures either were centered on the well or within 5 feet, usually to the west. Symmetrical uplift suggests that these fractures contained significant horizontal or sub-horizontal segments. Asymmetrical uplift and Uplift : Injection ratios substantially less than unity suggest that significant vertical or strongly sub-horizontal fracture segments are present. Fractures created above an approximate depth of 20 feet exhibited almost ideal horizontal uplift characteristics, whereas lower fractures generally displayed steeply dipping features. It is probable that some of the steeply climbing deeper fractures intersected and have partial interconnection with shallower horizontal fractures.

The quantification of interaction among fractures resulting from the fracture testing program provided the basis for well completion design. Fracture testing results indicated that strong fracture interconnections were limited to the E19, E26, and E34 fractures propagated from fractured recovery well FW-E. The results for these three fractures were consistent with very similar uplift patterns for each fracture. Testing results also indicated strongly independent fractures, such as J39. In general, however, the fracture testing results suggested that most fractures exhibited a degree of interaction between these two extremes. How the varying degrees of fracture interconnection will affect the ability to perform push-pull between adjacent fractures and/or adjacent wells cannot be determined until the remedial system is operated.

A total of 28 fractures associated with the 11 Phase 1A Area fractured recovery wells were identified in soil sample cores collected during the installation of confirmation borings. In addition to identifying sand type, the core samples also provided information about fracture thickness and orientation. Angles ranged from horizontal to vertical, as would be expected from the diverse orientations suggested by uplift. Variations between thickness of fractures revealed by exploration and uplift should be expected. Uplift is an averaging in which displacement of the surface results from the sum of local apertures at substantial distance below the ground surface. Thicknesses substantially greater than observed uplift can be explained only as an artifact of the sampling method. The absence of fractures at locations where uplift was recorded is believed to result from limitations of the various sampling methods used.

Synthesis of the available data leads to the following conclusions regarding fracture geometry:

- Hydraulic fractures formed within the upper 20 feet at Linemaster, i.e. within the upper till, generally have a horizontal form with dip angles of less than 15°.

- Fractures nucleated at greater depths generally have steep dip angles, possibly exceeding 75°. These fractures probably roll over and assume a more horizontal form at depths shallower than 20 feet.

The distribution of hydraulic fractures in the soils underlying the Phase 1A Area is non-uniform. Although the propagated fractures provide nearly complete lateral coverage, a less thorough vertical distribution of fractures was effected. Uplift characteristics indicated that horizontal or sub-horizontal fractures were created at depths less than 20 feet. In contrast, the deeper fractures exhibited uplift patterns indicative of sub-horizontal and steeply dipping fracture forms. Steeply dipping fractures propagated at depth result in limited lateral coverage, particularly at depth. The extensive regions between the fractured recovery wells at depth that are expected to be the most difficult to remediate.

7.0 DESIGN CONDITIONS FOR REMEDIATION SYSTEM COMPONENTS

This section provides the design criteria for the dewatering/SVE system components. These criteria were obtained from investigations, pilot tests, and modeling results described earlier in this report. The design criteria, where ranges were given, defaulted to the conservative value for the basis of system design and selection.

7.1 Phase 1A Area Dewatering

The initial phase of remediation is to dewater the overburden aquifer in the Phase 1A area. Automatic pneumatic pumps have been installed in each of the fractured wells and shallow bedrock well MW-10sb to depress the groundwater table throughout the Phase 1A area to allow vapor extraction throughout the overburden. Groundwater will be pumped into two separate water/product separators located on the mezzanine level in the garage, and drained into an equalization tank. From the equalization tank, the water will be pumped directly to the IRTS building for treatment and discharge.

The results of the hydraulic fracturing pilot test indicated a startup water flow rate of 1.5 gpm for up to 1.5 hours. During the pilot test, this rate dropped approximately 60 percent after one day of pumping. The sustainable short-term groundwater recovery rate was observed to be 0.05 and 0.2 gpm for FW-A and FW-B, respectively. The design basis for dewatering was established as 0.15 gpm per fracture well, and 1.0 gpm for the shallow bedrock well (MW-10sb).

The design criteria for the dewatering system components are as follows:

- Pneumatic Pumps - The design discharge flow rate per fracture well is 0.75 gpm at startup and 0.15 gpm long-term. These flows are consistent with those observed during the November 1995 pilot test. The discharge flow rate for the shallow bedrock well is estimated to be 1.5 gpm at startup and 1.0 gpm long term. Controllerless pneumatic pumps have been selected for this design. Pneumatic pumps were selected because no electrical centrifugal pump was identified that could operate continuously with the low flows expected.

- Discharge Piping - The design discharge piping is 0.75" diameter nylon tubing. This is the largest diameter size available for the pump model, and the nylon is chemically resistant to PCE. The standard discharge piping is ⅝". The discharge rate is 0.15 gallons per cycle. The number of cycles per minute depends on the recharge rate of the well. The diameter of the tubing is adequate to pass an equivalent flow of 1.5 gpm.

- Air Compressor (air delivery for the pneumatic pumps) - The total air consumption for fracture well pumps is 10 scfm based on data supplied by CEE, the pump manufacturer. Total air consumption for the shallow bedrock well pump is 2.0 scfm. The total design air flow delivery rate is 12 scfm at 70 psi. The specified equipment contains dual compressors and motors. Therefore, each can supply the requirements of the system resulting in 100 percent backup.

- Product/Water (TCE) Separators - Two product water separators are included. The sizes selected are standards produced by the manufacturer. The larger one can treat a maximum flow rate of 8 gpm. The discharge from fractured recovery wells FW-A, FW-G, FW-H, FW-I, and FW-J will be connected to this separator. The maximum projected flow rate from these fractured wells is 3.75 gpm. The flow from FW-E, the FW-F cluster, and shallow bedrock well MW-10sb will be directed to the second separator, which can treat a maximum flow rate of 5 gpm. The maximum flow rate from these wells is projected at 3 gpm.

- Transfer Pumps - There are two transfer pumps. One pump will transfer water accumulated in the air/water separator to the DVE equalization tank. The design flow rate for this transfer pump at the expected operating conditions is 8 gpm. The design flow originally was determined when the groundwater extraction well pumps discharged directly to the air/water separator. Subsequently, the extraction well pumps discharges were connected directly to the product/water separators. The capacity of the transfer pump likely is greater than necessary but the cost difference between the pump specified and smaller capacity pumps is insignificant. Consequently, the largest of the three transfer pump models initially specified was retained. The transfer pumps is sized to accommodate peak startup flows from the connected wells. A progressing cavity pump was selected for this design because the flows are low and the suction lift is high (vacuum).

The second transfer pump delivers water from the DVE equalization tank to the existing IRTS equalization tank. The capacity of this centrifugal pump is 20 gpm. the maximum anticipated flow is approximately 8 gpm.

- DVE Equalization Tank - The minimum tank capacity provides a 30 minute residence time at a maximum combined flow rate from both water/product separators. The thirty minute maximum total flow is approximately 205 gallons. The design tank volume is 275 gallons with a working volume of approximately 225 gallons.

7.2 Dewatering/SVE Treatment

Once the water table has been depressed to an appropriate level, the second phase of remediation will begin. The second phase of Phase 1A Area remediation is to apply a vacuum to individual fractures in each of the fractured wells to recover soil vapor within the vadose zone and increase the rate of groundwater withdrawal. To increase the soil vapor recovery rates, air will be supplied to some of the unsaturated fractures while adjacent fractures are under vacuum (push-pull air transfer).

The extracted groundwater will be handled and remediated as described in Section 7.1.

Air injection and extraction pipe sizes were selected to minimize head loss. In the areas where the highest TCE concentrations are known or expected, the soil vapor extraction piping will be nylon, which is chemically resistant to TCE. The remaining piping is PVC.

The extracted soil vapor will be discharged to an air/water separator. The recovered water fraction will be discharged to the equalization tank and directly pumped to the IRTS for treatment and discharge. The vapor fraction will proceed through a heat exchanger and into two activated carbon vessels. The purpose of the heat exchanger is to decrease the relative humidity and increase the effectiveness of the vapor phase carbon. The activated carbon will remove VOCs from the air stream. The treated air will then be exhausted to the atmosphere via ducts above the roof of the building by a vacuum blower located downstream of the carbon filters.

The design criteria for the SVE system components are as follows:

- Air Injection Blower - The design total injection air rate is 280 scfm for the fractured wells. The injection rate for each fracture well is a maximum of 40 scfm at 9 psig. This rate matches the design vapor extraction rate. The rate can be decreased, however, using the variable frequency drive. A heavy duty rotary blower was selected for this design.

- Vacuum Blower - The design total vacuum capacity is 360 scfm at 15" Hg (12" Hg at the well head) for the fractured wells, one shallow bedrock well, and trench (for on-site soil stockpile remediation). A heavy duty rotary blower was selected for this design. To increase blower motor efficiency, a variable frequency drive will be used to control motor speed and support operation over potentially, a wide range of conditions. As discussed in Section 5.0, the design air flow rate is 40 scfm including a safety factor of two. The vacuum was selected to develop an approximate one pore volume change per day and allow the use of a standard blower. At the design conditions, the blower will operate at approximately 36 HP. The 50 HP motor provided will allow the blower to operate at flow rates approaching 500 scfm.

- Air Injection Heat Exchanger - A heat exchanger is required for the air injection blower to reduce the discharge temperature below 150° F. At temperatures above 150° F, the PVC piping may begin to deform. The design conditions will result in a reduction of the discharge air temperature to 120° F. An air-to-air heat exchanger was selected for this design. The heat exchanger has the capability of reducing the temperature from 240°F to 116° F at a flow rate of 560 scfm.

- Vacuum Blower Heat Exchanger - A heat exchanger is required to increase the temperature of the inlet air upstream of the carbon filters. This will serve to increase the efficiency of the carbon. Heat from the air discharged from the vacuum blower will be used to heat incoming air using an air-to-air heat exchanger. The design temperature increase is 20° F. The heat exchanger has the capability of increasing the air temperature from 60 to 80° F at a flow rate of 500 scfm.

- Activated Carbon - The VOC loading will vary over time, especially as dewatering of the overburden progresses. As a result, the influent VOC concentration will change over time. The calculations from the air operating permit application are included in Appendix H. The concentration of VOCs in the air streams from the various extraction wells at the start of operation of the system was developed from concentrations of VOCs in the liquid phase. The groundwater VOC concentrations provide a conservative representation of the average VOC concentrations in the soils adjacent to the open or screened interval at each well. We suggest that the groundwater data represent concentrations of VOCs in the soil that will be subject to vapor extraction upon dewatering of the deeper soils. Equivalent vapor concentrations for each well and compound were calculated using the appropriate Henry's Law constant at 15°C. These concentrations then were converted to a volumetric basis.

  Equilibrium concentrations will be present only initially due to mass transfer limitations. Therefore, the equilibrium loading rate should be modified. Croise, et. al. have shown that VOC mass transfer rapidly becomes limiting and have proposed correlations based upon the number of pore volumes (PV) of air removed from the soil. This diffusion limitation on rates of soil remediation also has been discussed and modeled by Maroto. In general, for the first two to five pore volumes of air, the VOC concentration decreases to about 10 percent of the equilibrium value due to dispersive mixing in the macro-pores. After the two to five pore volumes, the concentration decreases from the 10 percent level according to the relationship: $(PV)^{-0.8}$. Thus, after about 1,000 pore volumes, the vapor-phase VOC concentration is 10^{-3} of the equilibrium value. Since 1,000 pore volumes of air could be removed from the shallow till within several days, a dilution attenuation factor (DAF) of 0.001 would be appropriate to apply to the total mass removal rate. We have suggested a more conservative dilution attenuation factor of 100 so the air loading for each compound should be divided by 100. The references have been provided previously.

This method of estimating vapor-phase VOC is very conservative and overstates the likely VOC concentration. The method was used to demonstrate that there would be no adverse air discharge impacts. Based on the air operating permit application rates, the effective first year VOC removal rate was estimated at approximately 15 lb/hr. This VOC discharge rate would result in a carbon utilization rate of approximately 700 pounds per day.

For the purposes of the SVE Air Operating Permit Application only, we calculated the potential VOC emission rates for the worst case startup condition at approximately 31 lb/hr. This estimate assumes the highest VOC concentration and that all the wells are producing maximum air flow rates.

When actual operation of the system begins, air flow rates will be adjusted consistent with VOC concentrations. Also, the moisture content of the soil will be high during the initial SVE operation further reducing the likelihood that very high vapor phase VOC concentrations will be observed.

- <u>Instrumentation</u> - Vacuum and air injection flow and pressure will be measured for each well and for the system using Magnehelic^{tm} direct reading gages. The ranges of the gages were determined by the availability of standard instruments or as developed during the design process. The discharge water flow rate from each fracture well and the well MW-10sb will be measured using in-line, rotary vane, totalizing flow meters. The flow range for these meters is 0.15 to 20 gpm. Total water flow to the IRTS will be measured using an inline, rotary vane, totalizing flow meter.

The instrumentation and an on-line VOC analyzer will provide the capability of continuous monitoring of the air flow rate and the concentration of VOCs from each fractured well. There will be a connection from the vacuum line from each fractured well to the on-line gas chromatograph. Periodically, approximately twice per day, an air sample will be withdrawn, in succession, from each fractured well. This sample will be analyzed for VOCs and the results stored in a PC dedicated to the remedial system. With an approximate 30 minute analytical time, approximately two samples per day could be analyzed form the 13 fractured wells.

To compliment the data collected with the GC, continuous data will be recorded from the vapor flow meters and pressure gauges. These data also will be stored in the dedicated PC. Examination of these data will allow an estimation of the mass of VOCs, and individual VOC compounds, removed during a specified time period. These data also will facilitate decisions on modification of system operation.

8.0 SURFACE BARRIER

Upon completion of the piping installation a surface barrier will be constructed over the Phase 1A area. The purpose of the surface barrier will be to inhibit rain and snow melt infiltration. It's construction also will reduce leakage of air injected into the fractures and vacuum short circuiting. The areal extent and proposed cross section are shown in the sketches included in Appendix I.

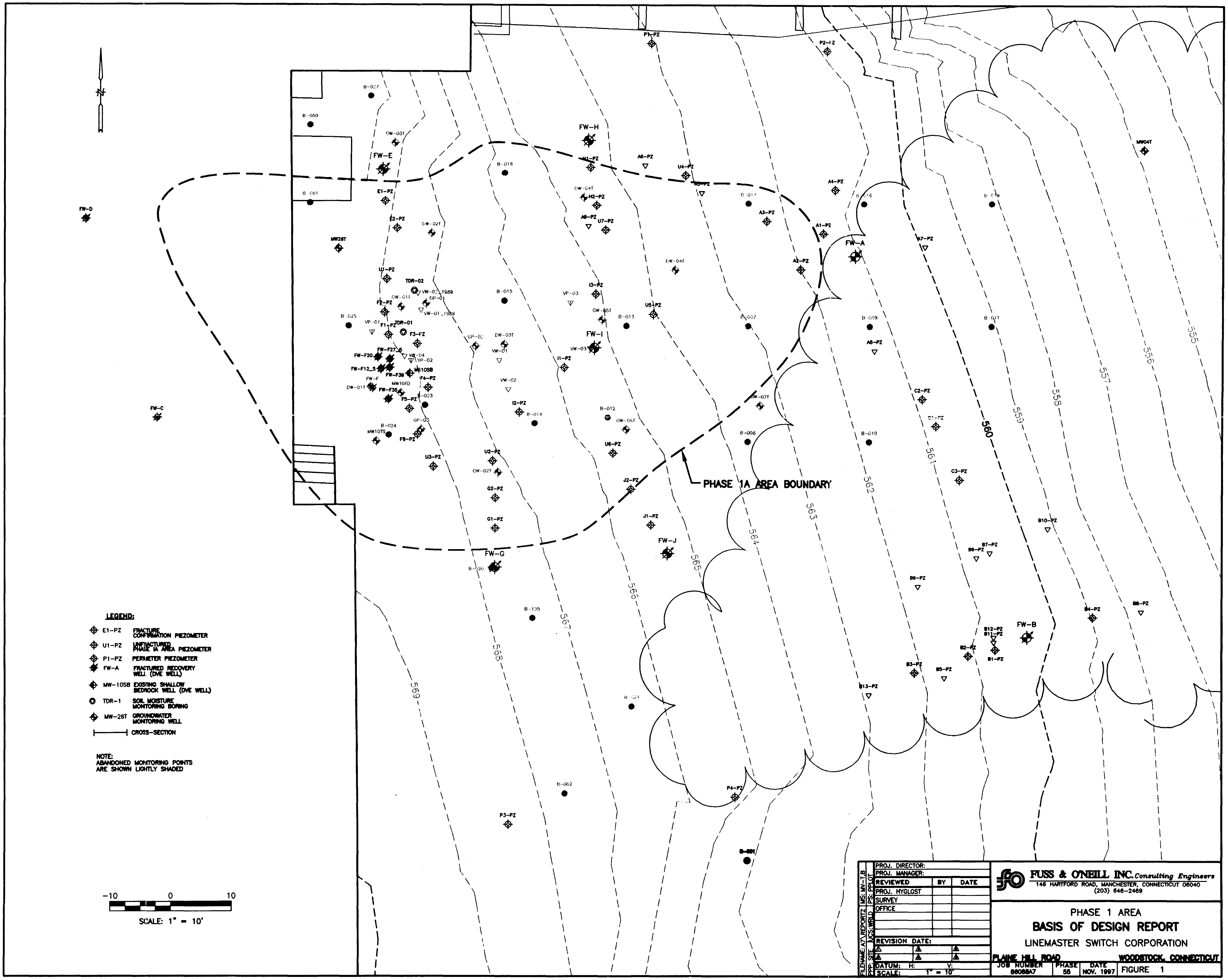
The barrier currently proposed barrier would consist of a geosynthetic fabric, with a permeability of a least 1×10^{-9} cm/sec, laid on the ground surface after it has been graded and cleared of rocks. A six inch drainage layer of sand would be placed on top of the fabric and graded. A final layer of topsoil approximately six inches thick will be placed above the sand and seeded.

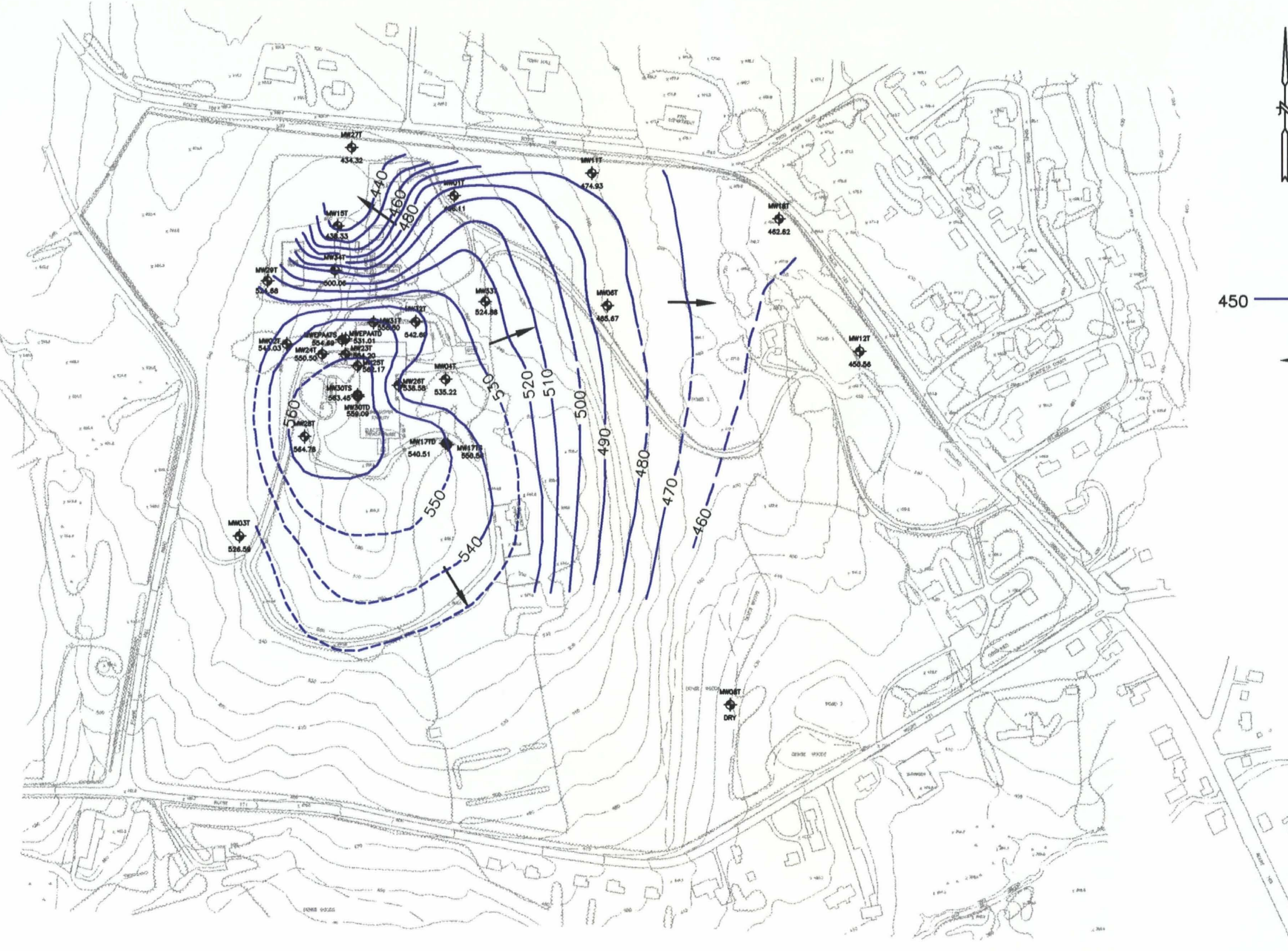
The geosynthetic liner fabric will be anchored at the building walls and seals will be installed around the wells, piezometers, and trees as required to prevent water infiltration. We will be adding two design drawings that will incorporate these sketches and details on anchoring the barrier and penetrations.

9.0 DESIGN CONFORMANCE TO SITE CONDITIONS

The proposed dewatering/soil vapor extraction system design takes advantage of existing site conditions. The following site conditions have been incorporated into the design :

- The proposed dewatering/soil vapor extraction system will be housed in a garage located adjacent to the Phase 1A Area.
- The two product/water separator tanks will be located on a five-foot high mezzanine to allow gravity flow of water to the equalization tank on the garage floor.
- Water effluent will be discharged to the nearby IRTS building for treatment, making use of the existing groundwater treatment system. The design flow rate of the existing IRTS system is 125 gpm. It currently operates at approximately 60 gpm. The maximum instantaneous increase would be 20 gpm from the DVE equalization tank over a few minute duration.
- Contaminated soils excavated as part of trenching activities will be treated on-site in the Phase IA Area. Remediation of stockpiled contaminated soil has been accounted for in the sizing of the vacuum blower by including a contribution of 40 scfm from the horizontal trench adjacent to the retaining wall.
- Air and water piping are sloped a minimum of 0.5% for drainage towards each well.
- Each fractured well vault is equipped with a heater, and water lines are buried a minimum of 48" to prevent freezing of water lines.


N
PHASE 1A AREA BOUNDARY
FW-A
FW-B
FW-C
FW-D
FW-E
FW-G
FW-H
FW-I
FW-J
555
556
557
558
559
560
561
562
563
564
565
566
567
568
569
LEGEND:
E1-PZ FRACTURE CONFIRMATION PIEZOMETER
U1-PZ UNFRACTURED PHASE IA AREA PIEZOMETER
P1-PZ PERIMETER PIEZOMETER
FW-A FRACTURED RECOVERY WELL (DVE WELL)
MW-10SB EXISTING SHALLOW BEDROCK WELL (DVE WELL)
TDR-1 SOIL MOISTURE MONITORING BORING
MW-26T GROUNDWATER MONITORING WELL
CROSS-SECTION
NOTE:
ABANDONED MONITORING POINTS ARE SHOWN LIGHTLY SHADED
-10 0 10
SCALE: 1" = 10'
PROJ. DIRECTOR:
PROJ. MANAGER:
REVIEWED BY DATE
PROJ. HYGLGST
SURVEY
OFFICE
REVISION DATE:
DATUM: H: V:
SCALE: 1" = 10'
FUSS & O'NEILL INC. Consulting Engineers
146 HARTFORD ROAD, MANCHESTER, CONNECTICUT 06040
(203) 646-2469
PHASE 1 AREA
BASIS OF DESIGN REPORT
LINEMASTER SWITCH CORPORATION
PLAINE HILL ROAD
WOODSTOCK, CONNECTICUT
JOB NUMBER 86088A7
PHASE 56
DATE NOV. 1997
FIGURE 1



LEGEND

MW-03T — OVERBURDEN (TILL) MONITORING WELL

526.59 — GROUNDWATER ELEVATION (FT.NGVD) NOVEMBER 3, 1997

450 — GROUNDWATER ELEVATION CONTOUR (DASHED WHERE INFERRED)

INFERRED DIRECTION OF GROUNDWATER FLOW

NOTES:

AT WELL CLUSTERS – THE GROUNDWATER ELEVATION AT THE SHALLOW WELL WAS USED FOR CONTOURING.

WELL MW-26T WAS NOT USED FOR CONTOURING BECAUSE IT IS AN ANGLED WELL SCREENED BENEATH THE BUILDING.

TOPOGRAPHIC FEATURES, SHOWN HEREON, WERE PREPARED IN ACCORDANCE WITH CLASS T-3 STANDARDS.

AERIAL PHOTOGRAPHY BASED ON 3-23-86 FLIGHT BY AERO GRAPHICS CORP. COMPILED BY AERIAL DATA REDUCTION ASSOC. CONTOURS BASED ON CONN. GEODETIC SURVEY STATION 1992 HAVING AN ELEVATION OF 567.141. (NGVD 1929).

HORIZONTAL DATUM BASED ON C.G.S. (NAD 1927).

ALL MONITORING WELLS HAVE BEEN FIELD LOCATED AND/OR VERIFIED HORIZONTALLY AND VERTICALLY.

CULTURAL FEATURES SOUTHEAST OF INTERSECTION OF RTE. 169 AND RTE. 171 ARE BASED ON PRELIMINARY PLANS FOR SOUTH WOODSTOCK SANITARY SEWER SYSTEM PREPARED BY CEE, INC., WEST HARTFORD, CONNECTICUT.

-300 0 300 600 900

SCALE: 1" = 300'

NOT RELEASED FOR CONSTRUCTION

FIGURE 2

FUSS & O'NEILL INC. Consulting Engineers
146 HARTFORD ROAD, MANCHESTER, CONNECTICUT 06040
(203) 646-2469

PROJ. MGR.: TLW
DESIGNER: SJR
FN: J:\A7\1197GWT
PPP:

UNCONSOLIDATED DEPOSITS GROUNDWATER CONTOUR MAP – NOVMEBER 3, 1997

BASIS OF DESIGN REPORT

LINEMASTER SWITCH CORPORATION

PLAINE HILL ROAD — WOODSTOCK, CONNECTICUT

DATE: MARCH 1998 — JOB NO: 86088A7

APPENDIX A

APPENDIX A

OVERBURDEN GEOLOGY AND PHYSICAL TECHNICAL MEMORANDUM

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM

LINEMASTER SWITCH CORPORATION
WOODSTOCK CONNECTICUT
FEBRUARY 1996

Prepared by:	Timothy L. Whiting, Jr. Project Hydrogeologist	Date: 2/23/96
	Frederick A. Mueller, P.E. Project Environmental Engineer	Date : 2/23/96
Project Manager:	David L. Bramley, P.E. Senior Environmental Engineer	Date : 2/28/96
Project Director:	Christopher R. Klemmer, P.E. Vice President	Date : 2/28/96

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Corres.

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM

TABLE OF CONTENTS

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM

TABLE OF CONTENTS CONTINUED

1.0 INTRODUCTION

The purpose of this Technical Memorandum is to provide a comprehensive summary of the Linemaster Switch Corporation Site (Linemaster) overburden geology and physical characteristics and related interpretations. The timing of the submittal of this Technical Memorandum is designed to enable agency concurrence on site conditions prior to submittal of the Conceptual Design Report. The data and interpretations presented in this Technical Memorandum may be revised, if necessary, and will be included in the Conceptual Design Report.

The presentation and interpretation of hydraulic conductivity and air permeability data are the primary focus of this Technical Memorandum. However, to facilitate this discussion, this Memorandum first provides a summary of the overburden geology and the related soil physical characteristics.

2.0 GEOLOGY

The geology of the unconsolidated deposits in the vicinity of the former dry well has been characterized based on the descriptions of subsurface soil samples as recorded in the boring logs prepared for all borings and monitoring wells installed at the Linemaster Site. Boring and monitoring well locations are shown on Figures 2.1A and 2.1B.

Logs for borings and wells installed prior to and during the Remedial Investigation (RI) were presented in appendices in the Remedial Investigation Report (F&O, 1992) and therefore have not been included herein. Boring logs prepared for monitoring wells and borings installed during post-RI investigations were included in the Zone 1 Delineation Report (F&O, 1996b).

Overburden deposits in the vicinity of the study area have been mapped as glacial till (Randall et al., 1966). During previous drilling investigations at the site, till deposits have been encountered at all boring locations. Consistent with mapping by Randall et al. (1966), this material has been found to consist of a dense, compact, non-sorted and non-stratified mixture of clay, silt, sand, gravel, cobbles, boulders and angular rock fragments. The percentage of coarse-grained sediments (gravel, cobbles, etc.) present in the fine-grained matrix has been observed to vary both laterally and vertically within the till deposits.

2.1 Field Observations

Subsurface soils encountered during drilling in the vicinity of the manufacturing facility building are considered to represent undisturbed, native glacial till deposits, with the exception of a limited area of fill. Logs from several borings/wells drilled close to the facility building (i.e. DW-1t, DW-2t, B-12, B-25, B-27, B-28, B-50, B-51, and OW-3t) indicate that in that area, between three to seven feet of sandy fill is present overlying the native till deposits. These fill deposits generally are present above the seasonal high water table and therefore can be considered of little importance relative to groundwater flow.

No continuous sand or coarse-grained lenses have been observed in the vicinity of the manufacturing facility building. The lateral extent of a suspected grain-supported (matrix devoid) gravel or cobble zone found below an approximate depth of 45 feet below grade at the B1-PZ piezometer cluster is unknown. The term "grain-supported zone" is intended to describe an interval in which significant void space exists due to the fact that the area between individual grains (e.g. cobbles) is not occupied by the fine-grained matrix normally present at other locations. The presence of a cobbley interval at a depth of 42-44 feet in the adjacent FW-B boring is suggested by the lack of sample recovery in this interval; however, as no sample was recovered, the presence or absence of fine-grained matrix in this interval can not be determined. Gravel and cobbles were encountered in a fine-grained matrix at a depth of 45 feet in boring B2-PZ, the extent of this interval and the presence or absence of matrix is unknown at greater depths because this boring was terminated at a depth of 46 feet below grade.

At lower elevations nearer the perimeter of the site, discontinuous sand lenses were found within the saturated till at monitoring wells MW-6t (15 ft) and MW-2t (46.5 ft). As discussed in the RI report, an increase in sand content of the till deposits is noted near the eastern and southern limits of the study area. In these areas, portions of the till may have been reworked and deposited as stratified drift. This interpretation is consistent with the mapping of coarser-grained stratified drift deposits in the Old Mill Brook valley south and east of the site (Randall et al., 1966).

Continuous soil sampling conducted during the 1995 Soil Fracturing Pilot Test drilling investigations in the area east of the Linemaster manufacturing facility building indicate that the glacial till consists of two till units: a dense, brownish-colored upper till and a dense, grayish-colored lower till. Field observations indicate that these two units are compositionally similar and are distinguished by subtle differences in grain size and color. Visually, in addition to the difference in color, the lower till appears slightly finer grained and contains less sand than the upper till.

The transition between these two units occurs at an approximate depth of 15 to 18 feet below grade in the Pilot Test area. The contact between these two units typically is sharp and may be erosional in nature. An interval marked by a higher percentage of coarse-grained material (i.e., gravel and cobbles) within the fine-grained matrix in the lowest portion of the upper till is suggested by increased drilling resistance and typically lower split-spoon sample recovery percentages.

2.2 Interpretation

Melvin et al. (1992a) identify and describe two distinct types of glacial till deposits that were deposited throughout Connecticut and Southern New England during separate Pleistocene glaciation episodes: an upper (or "surface") till unit and a lower (or "drumlin") till unit. Subglacial till deposits of lodgement origin are present in both the upper and lower tills. Previous differentiation of the glacial till deposits at the Linemaster Site as upper and lower till units intentionally has been consistent with the terminology used by Melvin et al. The lower till unit at the Linemaster site corresponds with the drumlin till, whereas the upper till at the site correlates to the surface till/mixed till zone of Melvin et al.

The field characteristics of the both the upper and lower till units present at the Linemaster Site suggest a lodgement till genesis. Although Melvin et al. note that thick "lower" till deposits in Connecticut most commonly are associated with drumlins, the bedrock high responsible for the topographic high upon which the Linemaster facility is located precludes the classification of this landform as a drumlin. However, these authors also note that thick "lower" till deposits also may be found as ramps on the north slopes of topographic highs. Such is the case at the Linemaster site, thick till deposits are present underlying the northern portion of the site where the depth to bedrock increases sharply to the north of the central bedrock high.

2.2.1 Naturally-Occurring Fractures

Although direct evidence is scarce, a network of naturally-occurring fractures is believed to be present in the glacial till deposits at the Linemaster site; for example, during the drilling of the B4-PZ piezometer cluster boring, a thin (<0.1 cm) fracture dipping 39 degrees downward to the west was noted at a depth of 15.5 feet in the B4-PZ piezometer cluster boring. This feature was distinguishable from the hydraulic fractures created during the Pilot Test by the absence of fracture sand or gel residue, the presence of discoloration (iron-oxidation) and traces of silt. The lack of natural fracture identification at other sampling locations or intervals may be a reflection of sample distortion during sampling or oversight rather than true absence. In the 1992 USGS publication entitled "The Stratigraphy and Hydraulic Properties of Tills in Southern New England", Melvin et al (1992a) note that jointing (or fracturing) commonly is well developed in drumlin tills in Connecticut and Southern New England and increases progressively upward in this unit.

2.3 Depth to Bedrock

Supplemental bedrock surface elevation data collected after completion of the RI report have been used to prepare a revised bedrock surface elevation contour map, Figure 2.2. This map was produced based on the following data:

- Known depths to bedrock at bedrock monitoring well locations;
- Depths to bedrock at borings and overburden monitoring wells where drilling refusal has been assumed to indicate the bedrock surface;
- Depths to bedrock reported in drillers' domestic supply well completion logs;
- Depths to bedrock based on boring and monitoring well logs prepared by Consulting Environmental Engineers following drilling activities completed for the design of the Woodstock municipal sewer; and,
- Approximate surface elevations of bedrock outcrops observed in the field.

Figure 2.2 indicates that the topographic high in the central portion of the Linemaster site is underlain by two roughly parallel north-south oriented bedrock surface highs. The westernmost of these topographic highs is observed to extend beneath the Linemaster manufacturing facility building. To the east of the building the bedrock surface slopes first to the east and then to the west with a trough-like bedrock surface low present slightly east of the area in which the Pilot Test was performed.

Immediately north of the manufacturing facility the bedrock surface dips to the north. Further to the north the bedrock surface dips dramatically and forms a prominent north-south oriented trough in the northwest portion of the site.

3.0 AQUIFER PARAMETER CHARACTERIZATION RESULTS

The following subsections provide a summary of all subsurface soil physical testing results for samples collected at the site. Samples have been selected for testing to enable characterization of physical conditions in both the shallow and deep portions of the overburden till deposits. Laboratory testing was performed to determine grain size distribution, dry bulk density, moisture content, specific density, and porosity. Horizontal hydraulic conductivity values were determined from analysis of in-situ testing. Vertical hydraulic conductivity and air permeability results also were determined by laboratory testing. Laboratory analysis was performed to determine total organic carbon concentrations.

3.1 Grain Size Distribution

The grain size distributions of 12 soil samples collected from the site were determined using standard sieves and hydrometers. These results are summarized in Table 3.1. Of the 12 samples tested, three were collected from the upper till and nine were collected from the lower till.

The results in Table 3.1 confirm the field findings that the till deposits predominantly consist of silt and clay, with substantial sand and varying amounts of gravel. The average grain size percentages for the upper and lower till are similar, with the exception of the percentage of gravel. The average percentage of gravel in the lower till (22%) is twice that in the upper till (11%). Although not readily apparent during drilling inspection, this information suggests that the sorting of the lower till is poorer and contains more coarse-grained sediments than the upper till.

The average sand content in the lower till (35%) is less than that of the upper till (39%); however, the average silt and clay content of the lower till samples (43%) is less than that found in the upper till samples (50%), which is in apparent contrast to field observations as discussed in Section 2.0. This apparent discrepancy may be related to the fact that field estimations of grain size distribution are based on volume estimates rather than weight percentages. Because laboratory determined grain size distributions are made on the basis of weight, the presence of only one or two large grains (i.e. gravel or pebble size) can strongly influence the grain size distribution.

TABLE 3.1
GRAIN SIZE ANALYSIS SUMMARY

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
FEBRUARY 1996

			SAMPLE DESIGNATIONS AND LOCATIONS													
			UPPER TILL SAMPLES				LOWER TILL SAMPLES									
		SAMPLE ID	415950919-06	415950928-53	415950925-31		373911205-117	415950821-56	373911204-97	373911205-111	415950823-65	415950920-19	415950926-45	415950809-26	415950817-48	
		LOCATION	C1-PZ	FW-A	FW-B	UPPER TILL AVERAGE	DW-3t	B-131	OW-2t	OW-1t	MW-33t	C1-PZ	FW-B	MW-32sb	MW-34t	LOWER TILL AVERAGE
		DEPTH	10.5'-11.0'	13.0'-13.5'	13.0'-13.5'		18.0'-20.0'	25.5'-26.0'	27.0'-29.0'	28.0'-30.0'	30.5'-31.0'	37.0'-37.5'	40.5'-41.0'	50.0'-52'	90.5'-91.0'	
PARTICLE CLASS	SIEVE SIZE	GRAIN SIZE RANGE (mm)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)
GRAVEL																
coarse	0.75 mm	75 - 19	0	2	13	5	10	32	27	5	9	29	4	3	9	14
fine	#4	19 - 4.8	2	8	8	6	10	6	8	10	5	7	10	14	4	8
		total %	2	10	21	11	20	38	35	15	14	36	14	17	13	22
SAND																
coarse	#10	4.8 - 2.0	3	5	3	4	4	1	5	3	3	2	4	5	5	3
medium	#40	2.0 - 0.43	9	10	9	9	8	7	7	10	10	8	10	10	11	9
fine	#200	0.43 - 0.08	30	25	23	26	24	19	19	26	27	17	26	22	23	23
		total %	42	40	35	39	36	27	31	39	40	27	40	37	39	35
SILT	HYDROMETER	0.08 - 0.0039	34	29	26	30	28	19	21	29	27	21	27	25	25	25
CLAY		< 0.0039	22	21	18	20	16	16	13	17	19	16	19	21	23	18
		total %	56	50	44	50	44	35	34	46	46	37	46	46	48	43

Notes: Grain size distribution percentages are by weight
Data summarized from Miller Engineering Lab Reports

In addition to the differences noted between the upper and lower till samples, it should be recognized that a high degree of variability in grain size distribution exists within both the upper and lower till units. This variability is to expected due to the inherent heterogeneity of glacial till deposits.

3.2 Porosity, Moisture Content, Bulk Density

Porosity, initial moisture content, and dry bulk density data are provided in Table 3.2. For comparison, average values have been determined for the soil samples collected from the upper and lower till. The upper and lower till sample averages are quite similar.

3.3 Total Organic Carbon

A total of 14 samples have been analyzed for determination of total organic carbon (TOC) concentrations. TOC results are provided in Table 3.3. These data will be used in the Conceptual Design Report for estimating contaminant mass removal rates.

4.0 HYDRAULIC CONDUCTIVITY

Hydraulic conductivity (K) values have been estimated by both laboratory and field (in-situ) hydraulic testing methods. All horizontal hydraulic conductivity (K_h) estimates are based on analysis of in-situ field data. All vertical hydraulic conductivity (K_v) values are based on laboratory testing results.

4.1 Scale Dependency of Hydraulic Conductivity Measurements

It is widely recognized that field hydraulic conductivity determinations (i.e., slug testing and aquifer pumping tests) typically are up to an order of magnitude greater than the values determined by laboratory testing. Direct comparison of field and laboratory K values may result in apparently conflicting conclusions. This phenomenon is attributed to the effects of scale dependency on hydraulic conductivity measurements and is discussed below by way of reference to several recent publications.

In the 1992 USGS publication entitled "Hydrogeology of Thick Till Deposits in Connecticut", Melvin et al (1992b, p. 27) discuss the observed differences in till hydraulic conductivity values as determined by field and laboratory methods and state the following:

> This discrepancy is ascribed to the till having both primary (intergranular or matrix) hydraulic conductivity and secondary hydraulic conductivity produced by fracturing or weathering. Only the primary hydraulic conductivity is measured by the laboratory tests, whereas the greater hydraulic conductivity due to secondary features, such as fractures, can be measured only by the field tests.

Bruner and Luttenegger (1994) presented the results of the comparison of in-situ and laboratory

TABLE 3.2
OVERBURDEN PHYSICAL CHARACTERISTICS

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
FEBRUARY 1996

Sample Location	Depth (ft.)	Sample Number	Laboratory	Dry Bulk Density (g/cm3)	Initial Moisture Content Gravimetric (%)	Initial Moisture Content Volumetric (%)	Specific Gravity	Porosity (%)
UPPER TILL SAMPLES								
C1-PZ	10.5 - 11	415950919-06	Miller Engineering	2.01	12.9		2.67	24.6
	10 - 12	415950919-06	D.B.Stephens	1.91	10.0	19.0		30.5
FW-A	13.0 - 13.5	415950928-53	Miller Engineering	2.03	12.5		2.64	23.0
FW-B	13.0 - 13.5	415950925-31	Miller Engineering	2.03	11.8		2.68	24.1
	12 - 14	415950925-31	D.B.Stephens	2.02	11.6	23.5		23.7
UPPER TILL SAMPLE AVERAGES				2.00	11.8	21.3	2.66	25.2
LOWER TILL SAMPLES								
DW-3t	18 - 20	373911205-117	Miller Engineering		12.8			25.0 *
B-131	25.5 - 26	415950821-56	Miller Engineering	2.02	9.6		2.68	24.5
OW-2t	27 - 29	373911204-97	Miller Engineering		11.1			25.0 *
OW-1t	28 - 30	373911205-111	Miller Engineering		10.2			25.0 *
MW-33T	30.5 - 31.0	415950823-65	Miller Engineering	2.02	10.3		2.73	25.9
C1-PZ	37.0 - 37.5	415950920-19	Miller Engineering	2.13	11.3		2.68	20.4
FW-B	40.5 - 41.0	415950926-45	Miller Engineering	2.08	12.1		2.67	22.0
	40 - 42	415950926-45	D.B.Stephens	1.96	11.3	22.2		25.9
MW-32sb	50.0 - 50.5	415950809-26	Miller Engineering	2.05	14.6		2.72	24.5
	50 - 52	415950809-26	D.B.Stephens	2.18	9.9	21.6		17.8
MW-34T	90.5 - 91.0	415950817-48	Miller Engineering	1.95	15.5		2.67	27.0
LOWER TILL SAMPLE AVERAGES				2.05	11.7	21.9	2.69	23.9

NOTES: Data summarized from Miller Engineering and D.B. Stephens Laboratory Reports.
* Indicates average of the 3 denoted samples

TABLE 3.3
TOTAL ORGANIC CARBON RESULTS

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
FEBRUARY 1996

SITE	DATE	DEPTH (ft)	TOTAL ORGANIC CARBON (mg/kg)
UPPER TILL SAMPLES			
B-27	06/11/91	9.0	1090
C1-PZ	09/19/95	11.0	2630
FW-B	09/25/95	13.0	2820
FW-A	09/28/95	13.0	1490
LOWER TILL SAMPLES			
B-29	06/13/91	19.0	1550
MW-26t	06/27/91	19.5	1150
B-129	09/25/95	21.0	1080
B-130	08/11/95	23.0	1940
B-131	08/11/95	28.0	2290
MW-23t	06/14/91	29.0	1190
MW-33T	08/23/95	31.0	2480
C1-PZ	09/20/95	37.0	3760
B-25	06/10/91	39.0	1100
FW-B	09/26/95	41.0	1100
MW-31T	08/02/95	56.0	2540

NOTE: Analyses performed by EPA Method Lloyd Kahn at IEA Laboratories, Monroe, Connecticut measurements of the hydraulic conductivity of fine-grained glacial tills in Iowa. Based on measurements of hydraulic conductivity performed by laboratory flexible-walled permeameter tests (both K_v and K_h), bailer tests (slug tests), and pumping tests in fine-grained glacial till, they concluded, in part, that:

- Very little difference was present between laboratory-measured values for geometric means of horizontal and vertical K: an anisotropy ratio (K_h:K_v) of 2.5 was calculated.

- The bulk (or secondary) K increased by several orders of magnitude as the tested volume of till increased.

- Bailer test (similar to slug tests) results were approximately two orders of magnitude greater than the lab-determined K_h.

- Pumping test derived K_h values were approximately one order of magnitude greater than the bailer test values.

- Increasing values of saturated K at larger spatial scales conceptually supported a double-porosity flow model for the fractured till.

In general, Bruner and Luttenegger (1994) conclude that fracture flow dominates the advection system in fractured tills common to Iowa and many other areas; therefore as different methods test larger volumes of sediment, there is more likelihood of encountering fractures, sand laminae, or other higher permeability features; and, for this reason, matrix values of K severely underestimate the bulk K. They recommend that a value three orders of magnitude greater than laboratory-measured K values should be used as an initial estimation of the bulk K.

Rovey and Cherkauer (1995) recently evaluated the scale dependency of hydraulic conductivity measurements by field testing the K of a carbonate aquifer using slug tests, pressure injection tests, pumping tests, and digital models. The effective test radii of these methods ranged from less than one meter to greater than 10,000 meters. They concluded that the hydraulic conductivity increases with effective test radius before becoming approximately constant at some distance and that their results were consistent with the scaling effects reported at seven additional sites in a variety of geologic media. They specifically noted that the distance after which K became constant was observed to be greatest in geologic units with greater secondary porosity, such as jointed (fractured) tills. Rovey and Cherkauer also concluded that scaling effects vary consistently with the type of geologic medium and degree of secondary porosity. Scaling effects are relatively minor in glacial outwash deposits, which are dominated by primary porosity, but are much greater in joint-dominated media (ie. karstic limestone, fractured tills).

Based on the above discussions, the following conclusions are reached:

- Laboratory hydraulic conductivity testing results provide an estimate of the "matrix" (or primary) hydraulic conductivity; and, due to the vertical orientation of the Linemaster samples when tested at the laboratory, the results are considered to reflect the vertical hydraulic conductivity of the samples;

- In-situ hydraulic testing (slug or pumping tests) may be considered representative of the "bulk" (or secondary) horizontal hydraulic conductivity of the till mass; and,

- From a groundwater flow and site-wide dewatering perspective, the most applicable hydraulic conductivity values are bulk values determined from field testing.

4.2 Horizontal Hydraulic Conductivity

In-situ slug tests have been conducted at 35 overburden monitoring wells at the site. The electronically recorded water-level recovery data from each well were analyzed using the Bouwer and Rice (1976) method for unconfined aquifers to determine the estimated horizontal hydraulic conductivity of the adjacent portion of the aquifer. Supporting analytical data and graphical solutions for slug tests that previously have not been formally submitted are provided in Appendix A.

Table 4.1 provides a summary of the horizontal hydraulic conductivity values determined from analysis of slug test data generated from each overburden monitoring well. For comparative purposes, geometric mean K values have been used rather than average K values because the recorded K values range over several order of magnitudes.

Geometric means were calculated for shallow till (screen midpoint less than 25 feet below grade) and deep till (screen midpoint greater than or equal to 25 feet below grade) monitoring wells located in the vicinity of the manufacturing facility building. Due to the paucity of monitoring wells screened in the upper twenty feet of the overburden sediments, a comparison of K values for the "upper till" and the "lower till" portions of the overburden aquifer is not practical. K values determined from slug tests conducted at overburden monitoring wells located distant from the manufacturing facility building are provided at the bottom of Table 4.1 for reference.

In general, K values generally are lowest in the vicinity of the central topographic high upon which the manufacturing facility is situated. The highest in-situ K_h values are associated with the monitoring wells located in the topographically lower eastern and southeastern portion of the study area where glaciofluvially reworked till deposits (stratified drift) may be present.

In the vicinity of the manufacturing facility, the results in Table 4.1 indicate that the in-situ K_h geometric mean of the shallow till is 0.015 feet per day (ft/day) with values ranging from 0.001 to 0.14 ft/day. In this same area, the K_h geometric mean calculated for the deep till is 0.003 ft/day, with minimum and maximum K_h values of 0.001 and 0.009 ft/day, respectively.

In the vicinity of the manufacturing facility building, the highest field-determined K_h value

TABLE 4.1
HORIZONTAL HYDRAULIC CONDUCTIVITY RESULTS

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDU
LINEMASTER SWITCH CORPORATION
FEBRUARY 1996

WELL	HORIZONTAL HYDRAULIC CONDUCTIVITY (FT/DAY)	SCREEN MIDPOINT DEPTH (FT BELOW GRADE)	SCREEN MIDPOINT ELEVATION (FT NGVD)
SHALLOW TILL (<25 FTBGS)			
MW-33t	0.001	20.0	516.0
MW-24t	0.001	23.0	546.7
OW-6t	0.003	21.5	544.0
OW-4t	0.005	21.5	543.7
MW-23t	0.006	21.0	549.3
C1-PZ-22	0.007	22.0	538.1
MW-EPA-Ats	0.01	23.0	545.4
OW-1t	0.01	23.5	544.6
MW-32t	0.014	21.0	540.7
OW-5t	0.02	22.5	543.0
OW-3t	0.02	24.0	543.9
DW-4t	0.03	20.5	544.0
DW-2t	0.03	22.5	545.1
OW-2t	0.07	22.5	545.0
MW-17ts	0.08	15.0	545.3
OW-7t	0.08	20.5	543.5
DW-3t	0.1	19.0	545.8
DW-1t	0.14	23.0	546.2
GEOMETRIC MEAN	0.015		
MAXIMUM	0.14		
MINIMUM	0.001		
DEEP TILL (>25 FTBGS)			
MW-31t	0.001	25.0	540.5
MW-4t	0.001	33.5	521.7
MW-10td	0.001	39.0	529.9
MW-EPA-Atd	0.003	58.0	510.0
MW-34t	0.003	107.0	452.7
MW-17td	0.005	37.0	522.9
MW-16t	0.007	25.0	545.0
C1-PZ-44	0.009	44.0	516.1
GEOMETRIC MEAN	0.003		
MAXIMUM	0.009		
MINIMUM	0.001		
OUTSIDE ZONE 1 AREA			
MW-2t	0.001	33.5	528.3
MW-1t	0.002	41.0	486.4
MW-3t	0.006	17.5	525.8
MW-15t	0.01	132.0	420.6
MW-11t	0.1	23.0	468.4
MW-6t	0.29	17.5	481.5
MW-18t	1	10.0	455.5
MW-12t	2	12.0	442.2
MW-8t	11.7	15.0	419.8
GEOMETRIC MEAN	0.08		

NOTE: DW- and OW-Series wells have fully penetrating screens.

(0.14 ft/day) was recorded at DW-1t, which is a fully penetrating well with a 40-foot screened interval. The slug test results from this well and the other OW- and DW-series wells may be less representative of in-situ conditions because of the comparatively small ratio of the slug-displaced volume to the interval of the aquifer being tested (35 to 40 feet) and the fact that discontinuous, more (or less) transmissive intervals may have a disproportionate influence on the water-level response. This point is evidenced by the 0.03 ft/day K_h value determined from a 25-day April, 1992 pumping test at DW-1t (See Appendix J in RI Report). Due to scale dependency as discussed in Section 4.1, pumping test-derived K_h values are considered more representative of the bulk horizontal hydraulic conductivity of an aquifer than slug test K estimates.

The data in Table 4.1 indicate that the K_h geometric mean of the shallow till is nearly one order of magnitude greater than the mean K_h of the deep till. A direct comparison of K_h values from paired shallow and deep overburden aquifer monitoring wells (i.e., MW-EPA-At and MW-EPA-Atd; MW-17ts and MW-17td) also indicates that K_h values decrease by approximately one order of magnitude from the shallow to deep till. This variability is believed to represent a true difference in the hydraulic properties of the younger, upper till unit and the older, lower till unit. In the vicinity of the manufacturing building, the variability of K values at different locations within the same portion of the overburden aquifer can be attributed to the compositional heterogeneity of the till deposits.

A MODFLOW groundwater flow model is currently being calibrated based on the responses observed during the recently completed Soil Fracturing Pilot Test. In support of that model, pumping test analyses were performed for each multi-level piezometer based on the drawdown and recovery water-level curves. Preliminary results indicate that the "effective" K_h of the upper and lower till units are 0.03 and 0.02 ft/day, respectively. (Note: the term "effective" is used to indicate that the K_h is influenced by the presence of sand-filled hydraulic fractures). When finalized, groundwater modeling results and these pumping test analyses will be included and discussed in the Conceptual Design Report.

4.3 Vertical Hydraulic Conductivity

A total of ten soil samples have been laboratory tested to estimate the hydraulic conductivity of the overburden aquifer. Due to the orientation of these samples at the time of collection and analysis, the results provided in Table 4.2 are considered vertical hydraulic conductivities (K_v).

The results for the "Till Block", a sample collected at a depth of three feet in the area east of the Linemaster facility, are included in Table 4.2; however, due to the fact that this sample was collected from the upper portion of the vadose zone and the probability of increased hydraulic conductivity due to root zone development, biologic activity and weathering, the results for this unsaturated sample were not considered during the calculation of the geometric mean K_v for the upper till.

The laboratory-determined K_v geometric means for the upper and lower till were 8.4×10^{-5} ft/day and 5.0×10^{-6} ft/day, respectively. These data indicate that the K_v of the upper till is

TABLE 4.2
VERTICAL HYDRAULIC CONDUCTIVITY RESULTS

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
FEBRUARY 1996

Sample Location	Depth (ft.)	Sample Number	Laboratory	Vertical Hydraulic Conductivity (cm/sec)	Vertical Hydraulic Conductivity (ft/day)
UPPER TILL SAMPLES					
ZONE 1 TILL BLOCK	3		Miller Engineering	1.06E-05 *	3.00E-02 *
C1-PZ	10.5 - 11	415950919-06	Miller Engineering	6.77E-08	1.92E-04
FW-A	13.0 - 13.5	415950928-53	Miller Engineering	1.64E-08	4.65E-05
FW-B	13.0 - 13.5	415950925-31	Miller Engineering	2.31E-08	6.55E-05
UPPER TILL SAMPLE GEOMETRIC MEAN				2.95E-08	8.36E-05
LOWER TILL SAMPLES					
B-131	25.5 - 26	415950821-56	Miller Engineering	3.08E-08	8.73E-05
MW-33T	30.5 - 31.0	415950823-65	Miller Engineering	8.76E-09	2.48E-05
C1-PZ	37.0 - 37.5	415950920-19	Miller Engineering	7.26E-09	2.06E-05
FW-B	40.5 - 41.0	415950926-45	Miller Engineering	1.67E-08	4.73E-05
MW-32sb	50.0 - 50.5	415950809-26	Miller Engineering	3.27E-08	9.27E-05
MW-34T	90.5 - 91.0	415950817-48	Miller Engineering	2.88E-08	8.16E-05
LOWER TILL SAMPLE GEOMETRIC MEAN				1.77E-08	5.02E-05

NOTES: Data summarized from Miller Engineering Reports
* Indicates value not used for mean calculation (Sample from unsaturated zone).

approximately one-third of an order of magnitude greater than the K_v of the lower till. The laboratory-determined K_v values are approximately two to three orders of magnitude less than the field-determined (or in-situ) K_h values discussed in Section 4.2. This relationship should be regarded in the context of the scale-dependency nature of hydraulic conductivity measurements as discussed in Section 4.1.

As noted in Section 4.2, a MODFLOW groundwater flow model is currently being calibrated based on the responses observed during the recently completed Soil Fracturing Pilot Test. When finalized, groundwater modeling results and related K_v values will be included and discussed in the Conceptual Design Report.

4.4 Summary

Based on the information presented in the two preceding sections, Table 4.3 has been prepared to summarize the vertical and horizontal hydraulic conductivity data. It should be noted that the geometric mean horizontal hydraulic conductivity values included in this table are based on slug-test results, although as discussed in Section 4.1, pumping test data generally provide a more accurate assessment of the regional bulk hydraulic conductivity. The decision to exclude the hydraulic conductivity value resulting from the DW-1t pumping test (April 1992) was made based on the fact that, due to the long screened interval of the well, this value actually represents an average hydraulic conductivity of the combined upper and lower till units. Due to the preliminary nature of the hydraulic fracturing Pilot Test data evaluation and the presence of hydraulic fractures, the hydraulic conductivity values determined from that test also have not been included in the summary table.

The field data indicate that the bulk horizontal hydraulic conductivity of the lower till is approximately one order of magnitude less than that of the upper till. Laboratory-derived results indicate that the vertical hydraulic conductivity of the upper till matrix is 1.7 times greater than the vertical hydraulic conductivity of the upper till matrix.

An estimation of anisotropy (K_h:K_v) from these data is inappropriate due the fact that the values for these two hydraulic conductivity parameters have been estimated by different methods.

5.0 AIR PERMEABILITY

The air permeability of the overburden till deposits has been determined by both laboratory methods and in-situ field testing. Soil samples were collected from discrete intervals at several borings and submitted for laboratory permeameter testing to estimate the air permeability of the upper and lower till units at the site. During the hydraulic fracturing Pilot Test, in-situ air permeability measurements made through pneumatic testing of the subsurface at control wells and fractured wells and then comparing the results to subsurface response models.

The air permeability results presented should be considered in the same context as the scale dependency of hydraulic conductivity measurements discussed in Section 4.1. The field and

TABLE 4.3
HYDRAULIC CONDUCTIVITY SUMMARY TABLE

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS
TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK CONNECTICUT
FEBRUARY 1996

Depth of Soil Zone	Horizontal Hydraulic Conductivity (ft/day)		Vertical Hydraulic Conductivity (ft/day)	
	Laboratory	Field[1]	Laboratory[2]	Field
0 to 5 feet (Upper Till)	N/A	N/A	3.0×10^{-2}	N/A
5 to 8 feet (Upper Till)	N/A	N/A	N/A	N/A
8 to 18 feet (Upper Till)	N/A	1.5×10^{-2}	8.4×10^{-5}	N/A
Greater than 20 feet (Lower Till)	N/A	3.0×10^{-3}	5.0×10^{-5}	N/A

Notes:

N/A- Data Not Available.

1 - The numbers presented in this column are the geometric means of the field-determined (slug test) horizontal hydraulic conductivities presented in Section 4.2

2 - The numbers presented in this column are the geometric mean of the laboratory determined vertical hydraulic conductivities presented in Section 4.3

laboratory data upon which the following discussions are based previously have been presented in the report entitled "Soil Fracturing Pilot Test Results" (Fuss & O'Neill, 1996a).

5.1 Laboratory Measurements

Measurements of the air permeability of three soil samples were performed by the D.B. Stephens Laboratory in Albuquerque, New Mexico using the Vapor Equilibration (VEQ) method. The samples were collected using the California-modified split-spoon sampling technique. The analyses performed provided an evaluation of the air permeability as a function of moisture content and also quantified the Klinkenberg Effect. In consideration of the orientation of the samples both when collected from the subsurface and when analyzed in the lab, the VEQ results represent estimations of the vertical air permeability.

Summary results of analyses performed by D.B. Stephens, including the moisture characteristics of the drainage curve, the air permeability of the soil sample at a pressure of one atmosphere, and the Klinkenberg Equation coefficients, are provided in Table 5.1.

5.1.1 Initial Moisture Content

The collection of the samples that were submitted to the D.B. Stephens laboratory for testing occurred near the end of a long drought period. The locations of the samples with respect to the water table surface and capillary fringe are summarized as follows:

- During the installation of the boring at fractured well FW-B on September 9, 1995, the water-level data from well C1-PZ indicated that the minimum depth to water (DTW) was 14.5 feet below grade. This level was measured before the development of the piezometer. Shortly after development of the piezometer, the levels dropped and the DTW was approximately 19 feet below grade. Based on these observations, the FW-B 10-12 foot sample was collected at a time when the water table was approximately five to nine feet lower than the sampled interval and that the sample was likely within the capillary fringe.

- During the installation of the boring at C1-PZ on September 9, 1995, wet (saturated) samples were observed beginning at a depth of 19 feet below grade. Since "moist" samples were first encountered at a depth of eight feet below grade, it is believed that the C1-PZ 10-12 foot deep sample was collected from the near the top of capillary fringe in the unsaturated zone at a distance of approximately seven to ten feet above the water table.

- The lower till samples at FW-B (40-42 ft) and MW-32sb (50-52 ft) were collected from depths well below the water table and are considered to have been saturated when collected.

The laboratory-determined initial air-filled porosities of the samples collected from the lower till are consistent with the field conditions observed at the time of sample collection. Any air-

TABLE 5.1
AIR PERMEABILITY TESTING RESULTS

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
FEBRUARY 1996

Sample Description and Location			Moisture Content and Porosity			Moisture Characteristics of Initial Drainage Curve		Air Permeability at 1 Atmosphere (*4) (cm ^ 2)	Klinkenberg Equation Coefficients (*3)	
Boring Location	Depth (ft.)	Sample Number	Initial Moisture Content (% vol.)	Calculated Porosity (% vol.)	Initial Air-Filled Porosity (% vol.)	Matric Potential (ft H20)	Moisture Content (%, vol)		Permeability Intercept (*6) (cm ^ 2)	Slope (atm*cm ^ 2)
FW-B	12 - 14 (Upper Till)	415950925-31	23.5%	23.7%	0.2%	10 110 22000	23.0 20.8 5.5	3.48E-11 8.03E-11 1.53E-10	2.39E-11 2.10E-10 3.11E-10	1.09E-11 -1.27E-10 -1.56E-10
C1-PZ	10-12 (Upper Till)	415950919-26	19.0%	30.5%	11.5%	--	--	--	--	--
FW-B	40 - 42 (Lower Till)	415950926-45	22.2%	25.9%	3.7%	10 30 110 21000	22.3 21.1 19.4 6.7	-9.79E-13 2.99E-11 1.5E-11 6.69E-11	-2.03E-11 1.20E-11 4.35E-11 1.82E-10	1.88E-11 1.78E-11 -2.76E-11 -1.13E-10
MW-32sb	50-52 (Lower Till)	415950809-26	21.6%	17.8%	0 (*5)	10 30 110 2600	30.2 27.5 20.8 12.0	1.90E-12 7.85E-11 1.94E-09 1.34E-09	8.43E-12 2.06E-10 2.83E-09 3.37E-09	-6.91E-12 -1.26E-10 -8.76E-10 -2.00E-09

NOTES:
1) Data summarized from D.B. Stephens Laboratory Report (Included in Appendix C)
*2) SHADED RESULTS: "Due to the poor linear fit, the intercept is unreasonable."
*3) The Klinkenberg Equation Coefficients describe best linear fit of the lab data and are used to obtain the air permeability of the matrix as follows:
Air Permeability = Slope *1/P + Permeability Intercept, where P is the absolute pressure of the air in the matrix measured in atmospheres.
*4) At standard temperature and pressure.
*5) The difference between the porosity and the initial moisture content was -3.8. Since this is not possible, the initial air filled porosity was assumed to be 0.
*6) The permeability intercept represents the permeability at infinite gas pressure. This permeability is representative of the matrix only.

filled porosity in the lower till samples is likely due to drainage that may have occurred during or following sample collection. In the upper till, the initial air-filled porosity varied from 0.2 to 11.5 percent. Based on the groundwater elevation at the time of sample collection, the upper till sample with the highest air-filled porosity was located the greatest distance above the water table.

5.1.2 Soil Moisture Characteristic Drainage Curve

A soil moisture characteristic curve describes the relationship between the soil's moisture content and the matric potential (or matric suction) of the soil. The moisture content is the fraction of the soil's volume that is occupied by water. The matric potential is a measure of the soil's negative pressure (suction) potential that results from the affinity of water to the soil matrix. Matric potential includes the effects of the capillary potential of the soil pores and the adsorption of water by particle surfaces. The matric suction of a soil causes the observed capillary rise (capillary fringe effect) in the overburden above a water table. The measurement of and the means for interpreting soil moisture characteristic curves are defined in Freeze & Cherry (1979) and Hillel (1982).

Field samples were dried in the laboratory using a pressure plate apparatus that allows points on the soil moisture characteristic curve to be determined. Since these points were determined while drying (or draining) the soil, the data points would lie on the soil moisture characteristic drainage curve. The resulting matric potentials, in feet of water suction, and the corresponding soil moisture contents are provided in Table 5.1. Examples of how these data can be interpreted are:

- The upper till sample from FW-B has a matric potential of 10 feet of water column and a moisture content of 23 percent, which predicts that the moisture content of the upper till at a distance of ten feet above the water table would be 23 percent. Since the lab determined total porosity of this sample was 23.7 percent, the air-filled porosity of the sample would be 0.7 percent (total porosity minus moisture content: 23.7% - 23% = 0.7%).

- The lower till sample from FW-B has a matric potential of 10 feet of water column and a moisture content of 22.3 percent. These data suggest that a portion of lower till would have a moisture content of 22.3 percent when the water table is lowered to a depth ten feet below the sample depth. Since the lab determined porosity of that till sample is 25.9 percent, the air-filled porosity of the sample is calculated to be 3.6 percent.

The above interpretations of matric potential and moisture content neglect other phenomena such as the effects of infiltration and condensation on the moisture content, and the effects of induced subsurface vacuums (and air flow) on matric potential. The relevancy of these phenomena to remediation of the Linemaster site and potential engineering controls are as follows:

- Infiltration effects may be reduced by the installation of an impermeable liquid barrier (cap) over the area of concern;

- Condensation of water vapor can occur in the subsurface when humid air is drawn into the subsurface. This effect will be most pronounced during the summer when the air typically is warm and humid and the subsurface is cooler. For example, when 80°F air at 80 percent humidity enters the subsurface and is cooled to 55°F, approximately 0.009 pounds of water will be condensed per pound of dry air entering the subsurface. If this phenomena is determined to be significant, then it may be possible to use engineering controls to reduce this effect by lowering the humidity of air entering the subsurface.

- The application of subsurface vacuums would help remove the moisture from the smaller soil pores of the soil matrix by increasing the matric potential (suction) on the soil. For instance, if the water table is maintained at 10 feet below a portion of till, and a vacuum equivalent to 10 feet of water column (0.7 atmospheres absolute or 8.8 inches mercury vacuum) is applied near that soil, then the total matric potential would be 20 feet.

5.1.3 Klinkenberg Equation Coefficients

The Klinkenberg Equation coefficients are experimentally determined values that quantitatively describe the Klinkenberg Effect (1941). The Klinkenberg equation describes how the permeability of a soil is dependent on the nature of the gas and takes into account the phenomenon of slip, which is related to the soil pore size and the mean free path of the gas molecules. The effect predicts that the gas permeability of a soil is an approximately linear function of the reciprocal mean pressure of the gas. The Klinkenberg Effect predicts an increase in the air permeability with decreasing mean air pressure and is most pronounced when the pore diameters are small with respect to the mean free path of the gas.

If the permeability is plotted with respect to the reciprocal mean pressure of the gas, then the relationship should be linear with a positive slope. The results of the laboratory analyses predicted, in many cases, a negative slope. In the cases where the predicted slope was positive, the slope was not significant when compared to the range of mean pressures that might be used in the remedial design (presumed to be between 0.6 and 1.0 atmospheres absolute). Based on these results, it does not appear that the Klinkenberg Effect will have a significant influence on subsurface air flow at this site.

5.1.4 Lab Data Summary

The VEQ results in Table 5.1 indicate that the vertical air permeability increases as the moisture content decreases. In the upper till sample, the variation in air permeabilities at different moisture contents was less than one order of magnitude. In the lower till sample collected at MW-32sb (50-52 ft.) the air permeability variation was three orders over the tested range of moisture contents. Air permeability variability was less than one order of magnitude over the tested range of moisture contents for the lower till sample collected at FW-B (30-32 feet) if the data with the poor linear fit are disregarded.

For the anticipated remediation conditions at the site, the matric potentials would likely be less than 100 feet of water column. In consideration of this assumption, the vertical air permeability at one atmosphere in the upper till would be in the range of 3 to 8 x 10^{-11} cm^2 and the vertical air permeability at one atmosphere in the lower till would be in the range of 2 x 10^{-12} to 2 x 10^{-11} cm^2. If the geometric mean of these permeability ranges are compared, the vertical air permeability of the lower till is approximately ten times less than that of the upper till. If only the lower till data associated with matric potentials of 30 feet are considered, then the deep till has a vertical air permeability that is approximately the same as the shallow till.

5.2 Field Measurements

Field measurements recorded during the soil vapor extraction phase of the hydraulic fracturing Pilot Test can be used to estimate the air permeability of the upper till, and in one case, the lower till as well. During the Pilot Test, vacuums were applied to the subsurface through the two fractured wells, FW-A and FW-B. Analysis of subsurface vacuum distribution recorded during these phases enables estimation of vertical air permeabilities. Pneumatic testing of the shallow upper till (ground surface to approximately five feet below grade) also was conducted by applying vacuums to the two control wells, D1-PZ and D2-PZ. Analysis of subsurface vacuum distributions during the control well tests enabled determination of both vertical and horizontal air permeability.

The rate at which soil vapor is extracted from the subsurface is a function of the air permeability, the geometry of the wells, and the applied vacuum. Analytical solutions of the flow of soil vapor to a well under a vacuum (e.g., Shan and others, 1992), which ignore the Klinkenberg Effect, are in the form:

$$\frac{Q}{P_a^2 - P_w^2} = f(k,g)$$

where Q is the volumetric discharge; P_a the absolute atmospheric pressure; P_w the absolute pressure at the well; and $f(k,g)$ is a function of the formation's air permeability, k, and geometry of the well and boundaries, g. At any given time, $f(k,g)$ is a constant that relates the vacuum on the well to the discharge. It is possible that $f(k,g)$ will change with time due to changes in k that accompany moisture content fluctuations or other factors.

5.2.1 Control Wells

During the soil fracturing pilot test, vacuum extraction tests were conducted on two, shallow control wells (D1-PZ and D2-PZ) to estimate the horizontal and vertical air permeability of the upper till. The wells were connected to the vacuum source and the rate of discharge was measured at several applied vacuums. Subsurface vacuums also were measured at the different piezometers in the vicinity of each well at one applied well vacuum during the test. Details of these tests are included in Section 7.2.7 of the Soil Fracturing Pilot Test Results report (F&O, 1996a).

Observed values of air flow and suction distribution were evaluated using two air flow models to estimate the air permeability. The first model used a Levenberg-Marquardt parameter estimation algorithm and the analysis in Shan and others (1992). The second model was AIR-2D by Joss & Baehr (1995b).

During both tests, the groundwater level was approximately eight feet below grade several yards away from the control well, therefore this depth was used as the distance to the lower impermeable boundary for each model. Table 5.2 provides a summary of the parameters and results of the two model analyses. Also included in the table are the subsurface vacuums observed in the field and predicted by each of the models using the estimated air permeability and the vapor extraction rate from the well.

While the first model (Shan et. al) accurately predicted the observed vacuums at the well edge and the closest monitoring points, it underestimated the most distant vacuum observations for both tests. The differences between the observed and the calculated subsurface vacuums were relatively small, with a small average error of less than 0.1 inches water for both control well tests.

The Air-2D model less accurately predicted the observed subsurface vacuum distribution. The air permeabilities were generally a factor of two higher than those calculated by the method of Shan et al. Both models predicted similar permeability anisotropy ratios (vertical:horizontal). Since the Shan et al. model better predicted the observed subsurface pressures, it's results are considered to be more representative of the field permeabilities and the results of the AIR-2D model will not be considered further.

The results of the parameter estimation indicate that k_h, the horizontal air permeability, ranges over a factor of two, from 1.0 to 2.2 x 10^{-9} cm^2 and that the vertical air permeability, k_v, ranges from 4.1 to 4.3 x $10^{-8}cm^2$. The ratio of k_v to k_h ranges from 18 to 41 for the two tests. Due to the nature of the tests and the depths of the control wells and monitoring points, these permeability estimates are likely representative of the shallow soils present within five feet of the ground surface.

5.2.2 Fractured Wells

During the vapor extraction portion of the fracturing pilot test, tests were conducted at each of the two fractured wells (FW-A and FW-B) to estimate the vertical air permeability. The wells were connected to the vacuum source and the rate of discharge was measured at several applied vacuums. In some tests, one or more fractures were opened to the atmosphere while others were connected to the vacuum. Measurements recorded during the tests allowed determination of both the rate of air extracted from the hydraulic fractures and the flow rate of passively injected air into the hydraulic fractures open to the atmosphere. Data presented in Tables 7.8 and 7.9 and Figures 7.18 and 7.19 of the Soil Fracturing Pilot Test Results report (Fuss & O'Neill, 1996a) were used in the analyses.

As discussed in the following subsections, fractured well Pilot Test data were analyzed using

TABLE 5.2
CONTROL WELL TESTING ANALYSIS

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
FEBRUARY 1996

TEST 1 (D1-PZ)

VAPOR EXTRACTION WELL MODEL INPUT PARAMETERS		
DEPTH TO BOTTOM OF SCREEN / GRAVEL PACK	(FEET)	6.2
DEPTH TO TOP OF SCREEN / GRAVEL PACK	(FEET)	5.2
DEPTH TO LOWER IMPERMEABLE	(FEET)	8.0
WELL DISCHARGE	(CFM)	1.78

RESULTS: ESTIMATED AIR PERMEABILITY		MODEL	
		SHAN ET AL.	AIR-2D
HORIZONTAL (RADIAL) kr	(CM ^ 2)	2.24E-09	3.12E-09
VERTICAL kv	(CM ^ 2)	4.06E-08	5.12E-08
kv:kr		18	16.4
AVERAGE RESIDUAL	(inches water column)	0.0378	0.091

MODEL PREDICTIONS OF SUBSURFACE VACUUM RESPONSE				
LOCATION		SUBSURFACE VACUUM		
		OBSERVED	CALCULATED	
DISTANCE (FEET)	DEPTH (FEET)	IN FIELD ("H2O)	BY SHAN ET AL. ("H2O)	BY AIR-2D ("H2O)
0.12	6.00	110	110	110
2.82	3.58	1.25	1.26	1.14
6.10	2.82	0.17	0.07	0.048

TEST 2 (D2-PZ)

VAPOR EXTRACTION WELL MODEL INPUT PARAMETERS		
DEPTH TO BOTTOM OF SCREEN / GRAVEL PACK	(FEET)	3.7
DEPTH TO TOP OF SCREEN / GRAVEL PACK	(FEET)	3.4
DEPTH TO LOWER IMPERMEABLE	(FEET)	8.0
WELL DISCHARGE	(CFM)	1.13

RESULTS: ESTIMATED AIR PERMEABILITY		MODEL	
		SHAN ET AL.	AIR-2D
HORIZONTAL (RADIAL) kr	(CM ^ 2)	1.04E-09	2.37E-09
VERTICAL kv	(CM ^ 2)	4.29E-08	9.85E-08
kv:kr		41	41
AVERAGE RESIDUAL	(inches water column)	0.0873	0.21

MODEL PREDICTIONS OF SUBSURFACE VACUUM RESPONSE				
LOCATION		SUBSURFACE VACUUM		
		OBSERVED	CALCULATED	
DISTANCE (FEET)	DEPTH (FEET)	IN FIELD ("H2O)	BY SHAN ET AL. ("H2O)	BY AIR-2D ("H2O)
0.12	3.58	104.6	104.6	107
2.82	5.71	0.40	0.42	0.21
4.82	2.82	0.26	0.022	0.007

both a one-dimensional and a three-dimensional model.

5.2.2.1 One-Dimensional Fractured Well Model

A one-dimensional model was developed to estimate the vertical air permeability of the upper till using simplified geometry for the subsurface conditions. The significant assumptions used in the model were:

- The vertical air permeability of the upper till does not vary over the area of the fracture and does not vary with depth. Since many of the test cases involve air flow from the ground surface where the till is subject to more weathering, the model may overestimate the vertical permeability of the shallow till that is less weathered at greater depths;

- The direction of air flow from the ground surface to the fracture on vacuum is vertical. This requires that the flow into the side of a fracture is negligible. Since these flows may not be negligible, this assumption may cause the model to overestimate the vertical air permeability of the upper till;

- Air flow from the ground surface to the fracture on vacuum occurs over the entire area of the fracture. This assumes that the fracture is completely dewatered. Since some of the deeper fractures may not have been completely dewatered, the actual dewatered area may be less than assumed, thereby causing the model to underestimate the vertical air permeability;

- The vacuum induced in the fracture is uniform and is the same as the vacuum measured at the fracture well head. Since there are likely some pressure losses through the plumbing, well screen, and sand-filled fracture, this assumption may cause the model to underestimate the vertical air permeability; and

- The fractures and ground surface are horizontal and uniformly spaced. Because field observations indicated that the distance between fractures was not uniform and that the fractures tended to rise and become shallower at greater distances from the fractured well, the model may overestimate the vertical air permeability of the upper till.

A summary of the one-dimensional model development is provided in Appendix B. This model was used to evaluate the following types of data collected during the SVE portion of the soil fracturing pilot test:

- Test modes where vacuums were applied to only one fracture at a time and the predominant air flow is from the ground surface to the fracture under vacuum. For these configurations the model parameters were the depth to the fracture, the vacuum applied to the fracture, the flow rate of vapor extracted from the well, and the area of the fracture under vacuum.

- Test modes where vacuums were applied to only one or more groups of fractures and air was passively vented into another fracture. For these configurations, the model parameters were the distance between the two fractures, the vacuum difference between the two fracture, the flow rate of vapor vented into the well open to the atmosphere, and the overlapping areas of the two fractures.

A summary of the Pilot Test data and the computed vertical air permeabilities for each of these tests is provided in Table 5.3. The variation of permeabilities in each test mode was relatively small and only the average air permeability computed for each test mode will be considered. The air permeabilities predicted by these analyses fall within the range of 3.8×10^{-10} to 9.0×10^{-9} cm^2.

- If only the data from test modes where there was air flow from the ground surface to fractures less than or equal to 18 feet below grade are considered, the predicted vertical air permeabilities are within the range of 1.5 to 9.0×10^{-9} cm^2. This relatively narrow range suggests that subsurface conditions in the upper till unit do not vary significantly in the vicinity of each fractured well.

- If only the data from test modes where there was air flow between fractures are considered, the predicted vertical air permeabilities are within the range of 5.8 to 13×10^{-10} cm^2. This range of permeabilities is lower than the vertical air permeabilities estimated from pneumatic tests where air flow was from the ground surface to the fractures.

- At FW-A there is a clear trend in decreasing vertical permeability with increasing depth. Due to the limitations of this simplified model, it is likely that some of this decrease may be attributed to the lack of dewatering at greater depths (resulting in an over estimation of the dewatered area in which air flow is possible) or to invalid model assumptions.

- An apparent trend in decreasing vertical permeability with increasing depth is suggested at FW-B.

5.2.2.2 Three-Dimensional Fractured Well Model

A three-dimensional model was developed using AIR-3D (Joss and Baehr, 1994) to replicate the subsurface pressure distributions observed in the upper till under the vacuum applied at the fractured wells during the soil fracturing Pilot Test. An advantage of this model is the capacity to consider the three-dimensional geometry of the hydraulic (sand-filled) fractures. This model is being refined; its results will be incorporated into the Conceptual Design Report.

5.2.3 Field Data Summary

The vertical air permeability estimates developed using field data are summarized in Figures 5.1 and 5.2. The data in these figures indicate the zone of soil over which the permeability estimate

TABLE 5.3
AIR PERMEABILITY CALCULATIONS

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
FEBRUARY 1996

GROUND SURFACE TO 8 FOOT DEEP FRACTURE AT FW-A

FRACTURE PILOT TEST MODE	FRACTURE AREA (ft2)	TRAVEL DISTANCE (ft)	APPLIED VACUUM ("H20)	RESULTANT AIR FLOW (scfm)	ESTIMATED AIR PERMEABILITY (cm2)	(ft2)
MISC A3	554	8	49.8	24.7	8.4E-09	9.0E-12
MISC A3	554	8	44	21.6	8.4E-09	9.0E-12
MISC A3	554	8	35	18.7	8.2E-09	8.9E-12
MISC A3	554	8	21	12.6	8.9E-09	9.5E-12
MISC A3	554	8	12.3	7.9	9.8E-09	1.1E-11
MISC A3	554	8	48	24.3	1.0E-08	1.1E-11
MISC A3	554	8	42.5	20.9	8.5E-09	9.2E-12
MISC A3	554	8	34.5	18.8	8.2E-09	8.9E-12
MISC A3	554	8	25.4	14.6	9.0E-09	9.7E-12
MISC A3	554	8	17.5	11	9.4E-09	1.0E-11
MISC A3	554	8	8.3	4.8	1.0E-08	1.1E-11
				AVERAGE	**9.0E-09**	**9.7E-12**
				STANDARD DEVIATION	7.5E-10	8.1E-13

GROUND SURFACE TO 18 FOOT DEEP FRACTURE AT FW-A

FRACTURE PILOT TEST MODE	FRACTURE AREA (ft2)	TRAVEL DISTANCE (ft)	APPLIED VACUUM ("H20)	RESULTANT AIR FLOW (scfm)	ESTIMATED AIR PERMEABILITY (cm2)	(ft2)
MISC A2	1115	18	82	7.05	1.7E-09	1.8E-12
MISC A2	1115	18	132	7.9	1.3E-09	1.4E-12
MISC A2	1115	18	163	9.5	1.3E-09	1.4E-12
MISC A2	1115	18	196	11.5	1.4E-09	1.5E-12
MISC A2	1115	18	199.4	10.8	1.3E-09	1.4E-12
MISC A2	1115	18	21.7	1.7	1.4E-09	1.5E-12
MISC A2	1115	18	46.5	4.5	1.8E-09	2.0E-12
MISC A2	1115	18	60.8	5.5	1.7E-09	1.9E-12
MISC A2	1115	18	86.5	7.4	1.7E-09	1.8E-12
MISC A2	1115	18	118	8.7	1.5E-09	1.6E-12
MISC A2	1115	18	144.5	9.9	1.5E-09	1.6E-12
MISC A2	1115	18	155.5	10.3	1.5E-09	1.6E-12
MISC A2	1115	18	84	7	1.6E-09	1.8E-12
MISC A2	1115	18	123.5	8.7	1.5E-09	1.6E-12
MISC A2	1115	18	160	9.4	1.3E-09	1.4E-12
MISC A2	1115	18	198	11.2	1.3E-09	1.4E-12
				AVERAGE	**1.5E-09**	**1.6E-12**
				STANDARD DEVIATION	1.8E-10	1.9E-13

TABLE 5.3
AIR PERMEABILITY CALCULATIONS

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
FEBRUARY 1996

GROUND SURFACE TO 28 FOOT DEEP FRACTURE AT FW-A

FRACTURE PILOT TEST MODE	FRACTURE AREA (ft2)	TRAVEL DISTANCE (ft)	APPLIED VACUUM ("H20)	RESULTANT AIR FLOW (scfm)	ESTIMATED AIR PERMEABILITY (cm2)	(ft2)
MISC A1	1398	28	73.5	1	3.3E-10	3.5E-13
MISC A1	1398	28	112.5	1.5	3.4E-10	3.7E-13
MISC A1	1398	28	177.5	2.5	4.0E-10	4.3E-13
MISC A1	1398	28	219	3.3	4.5E-10	4.9E-13
				AVERAGE	**3.8E-10**	**4.1E-13**
				STANDARD DEVIATION	4.9E-11	5.3E-14

GROUND SURFACE TO 8 FOOT DEEP FRACTURE AT FW-B

FRACTURE PILOT TEST MODE	FRACTURE AREA (ft2)	TRAVEL DISTANCE (ft)	APPLIED VACUUM ("H20)	RESULTANT AIR FLOW (scfm)	ESTIMATED AIR PERMEABILITY (cm2)	(ft2)
MISC B1	873	8	84	20.6	2.8E-09	3.0E-12
MISC B1	873	8	75.8	18.8	2.8E-09	3.0E-12
MISC B1	873	8	65	17.1	2.9E-09	3.1E-12
MISC B1	873	8	50.4	13.8	2.9E-09	3.2E-12
MISC B1	873	8	25.3	10.4	4.3E-09	4.6E-12
MISC B1	873	8	20.5	6	3.0E-09	3.3E-12
				AVERAGE	**3.1E-09**	**3.3E-12**
				STANDARD DEVIATION	5.3E-10	5.7E-13

GROUND SURFACE TO 13 FOOT DEEP FRACTURE AT FW-B

FRACTURE PILOT TEST MODE	FRACTURE AREA (ft2)	TRAVEL DISTANCE (ft)	APPLIED VACUUM ("H20)	RESULTANT AIR FLOW (scfm)	ESTIMATED AIR PERMEABILITY (cm2)	(ft2)
MISC B2	896	13	89.7	20.1	4.0E-09	4.3E-12
MISC B2	896	13	81	18.2	4.0E-09	4.3E-12
MISC B2	896	13	71	17	4.2E-09	4.5E-12
MISC B2	896	13	58	14.6	4.3E-09	4.6E-12
MISC B2	896	13	44	10.4	4.0E-09	4.3E-12
MISC B2	896	13	26	6.6	4.2E-09	4.5E-12
				AVERAGE	**4.1E-09**	**4.4E-12**
				STANDARD DEVIATION	1.3E-10	1.4E-13

TABLE 5.3
AIR PERMEABILITY CALCULATIONS

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
FEBRUARY 1996

FLOW BETWEEN 8 FOOT DEEP FRACTURE TO 18 FOOT DEEP FRACTURE AT FW-A

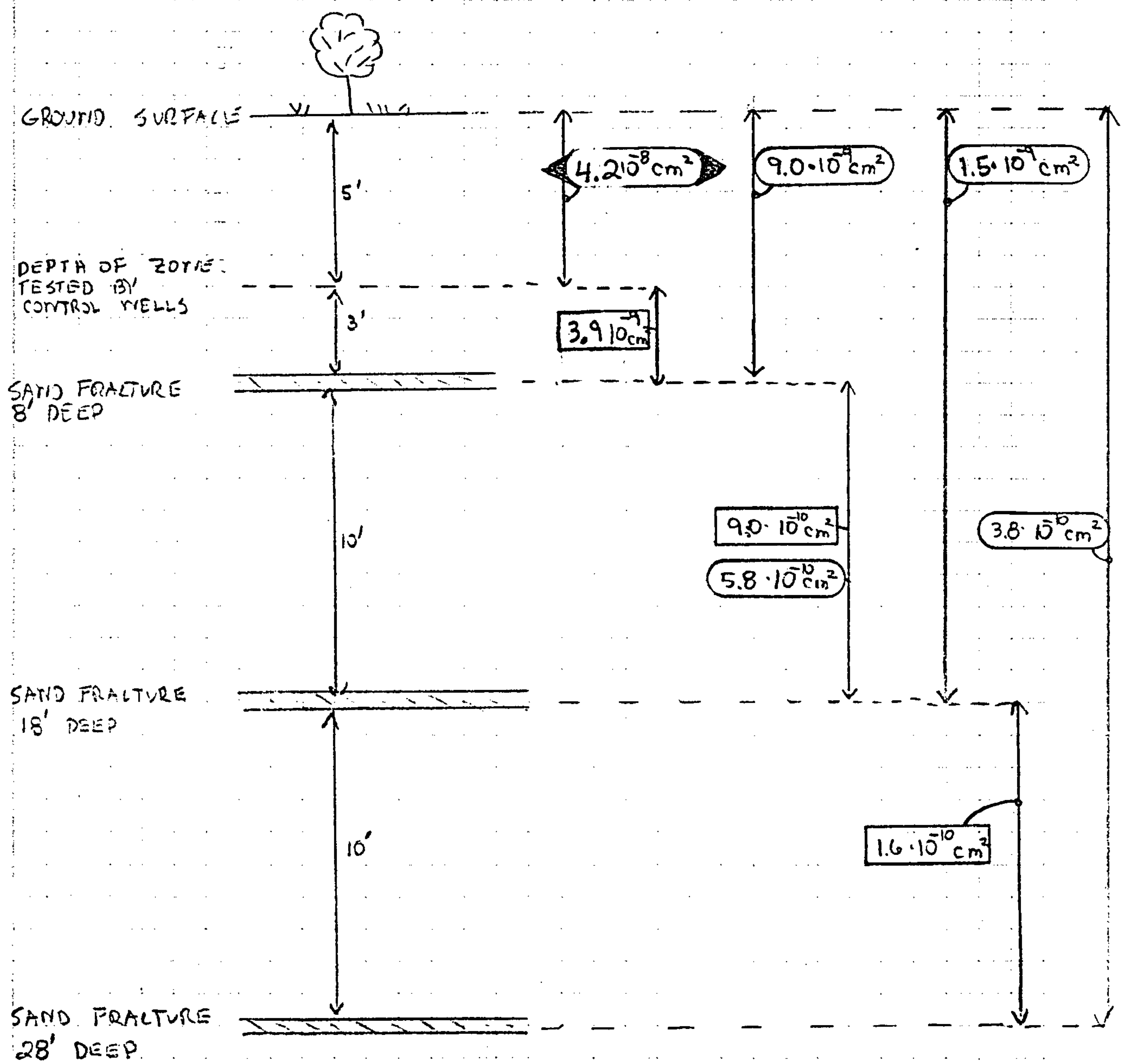
FRACTURE PILOT TEST MODE	OVERLAPPING FRACTURE AREA (ft2)	THICKNESS BETWEEN FRACTURES (ft)	DIFFERENCE IN FRACT. VACUUM ("H20)	AIR FLOW INTO OPEN FRACTURE (scfm)	ESTIMATED AIR PERMEABILITY (cm2)	ESTIMATED AIR PERMEABILITY (ft2)
TEST A1	414	10	151.5	3.9	5.0E-10	5.4E-13
TEST A2	414	10	42.5	1.65	6.5E-10	7.0E-13
				AVERAGE	**5.8E-10**	**6.2E-13**
				STANDARD DEVIATION	7.5E-11	8.0E-14

FLOW BETWEEN 8 FOOT DEEP FRACTURE TO 13 FOOT DEEP FRACTURE AT FW-B

FRACTURE PILOT TEST MODE	OVERLAPPING FRACTURE AREA (ft2)	THICKNESS BETWEEN FRACTURES (ft)	DIFFERENCE IN FRACT. VACUUM ("H20)	AIR FLOW INTO OPEN FRACTURE (scfm)	ESTIMATED AIR PERMEABILITY (cm2)	ESTIMATED AIR PERMEABILITY (ft2)
TEST B1	558	5	89.9	4.6	9.2E-10	9.9E-13
TEST B1	558	5	83	4.4	9.4E-10	1.0E-12
TEST B4	558	5	56	6.5	2.0E-09	2.1E-12
				AVERAGE	**1.3E-09**	**1.4E-12**
				STANDARD DEVIATION	5.0E-10	5.4E-13

FIELD Vertical AIR PERMEABILITY ESTIMATES AT FW-A

FIGURE 5.1



NOTES

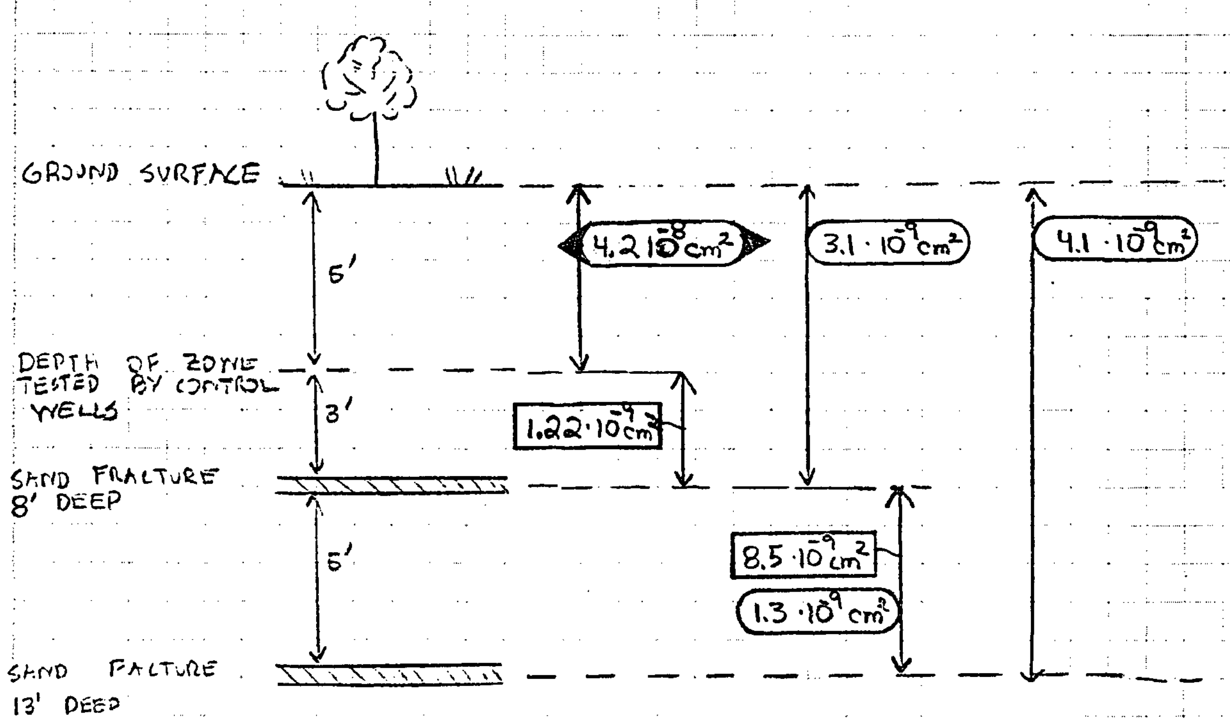
(#) The # is the permeability estimated by the "Control Well" Test

(#) - The # is the permeability estimated using the One-Dimensional Model and Field Data for vacuums and flows across the soil zone indicated.

[#] The # is the permeability computed based on the estimated permeabilities. See Sheet 2 for more details.

FIELD VERTICAL AIR PERMEABILITY ESTIMATES AT FW-B

FIGURE 5.2



NOTES

(diamond-ended rounded box #) The # is the vertical air permeability estimated from the results of the testing at the Control Well.

(rounded box #) The # is the vertical air permeability estimated using the One-Dimensional Model and the field data for vacuums and flows across the soil zone indicated.

$\boxed{\#}$ The # is the vertical air permeability computed based on the estimated permeabilities. The computation is performed as follows:

L_1 — K_1, L_2 — K_2 } K_{eq}

$$K_{eq} = \frac{L_1 + L_2}{L_1/K_1 + L_2/K_2}$$

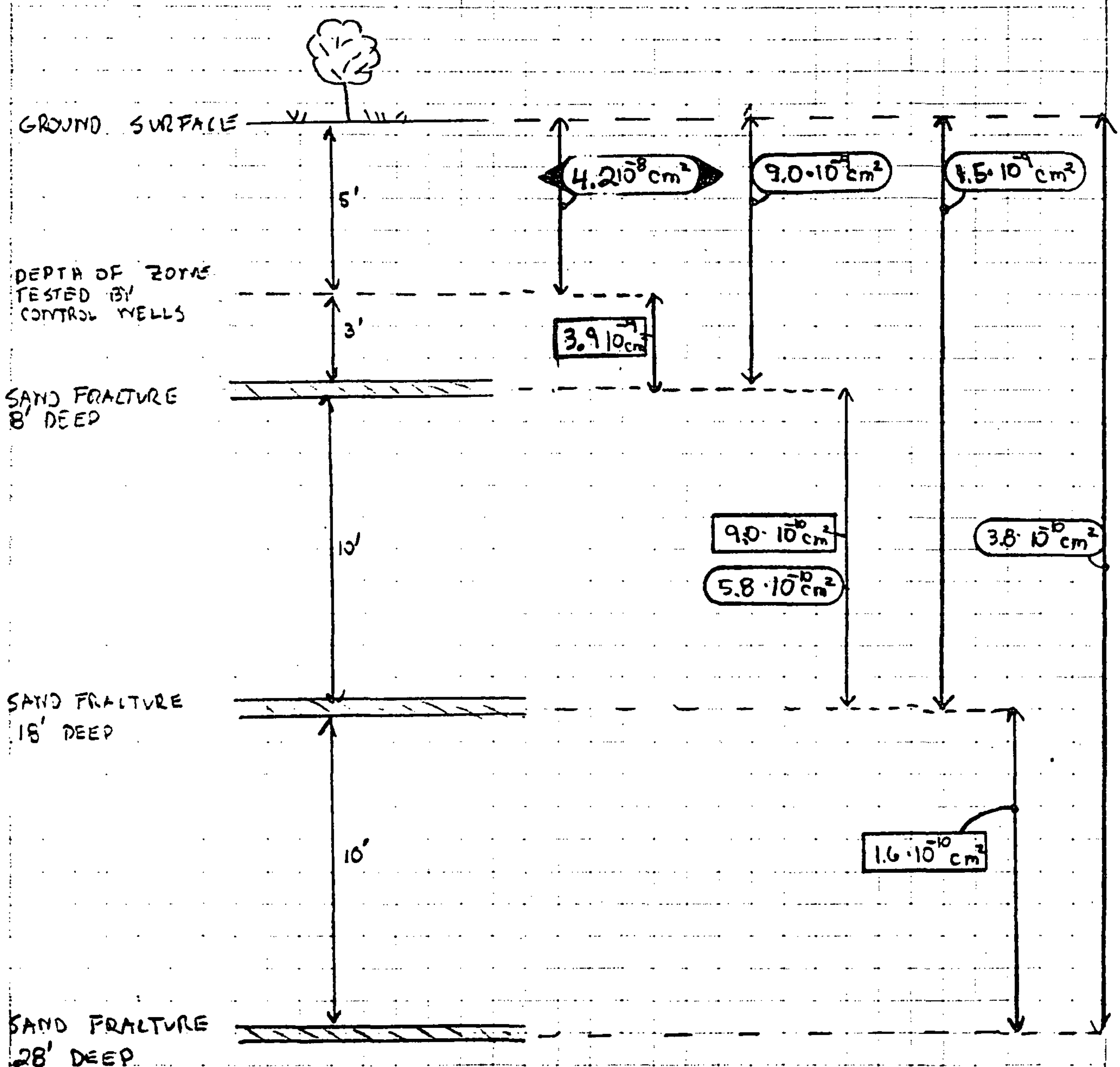
(Anderson & Woessner, 1992)

Typically K_{eq}, K_1, L_1, L_2 are known and K_2 is solved for.

Fuss & O'Neill Inc. *Consulting Engineers*	PREPARED BY	DATE	CHECKED BY	DATE	PROJECT NO
	FAM	2/20/96			86-088A

FIELD Vertical AIR PERMEABILITY ESTIMATES AT FW-A

FIGURE 5.1



NOTES

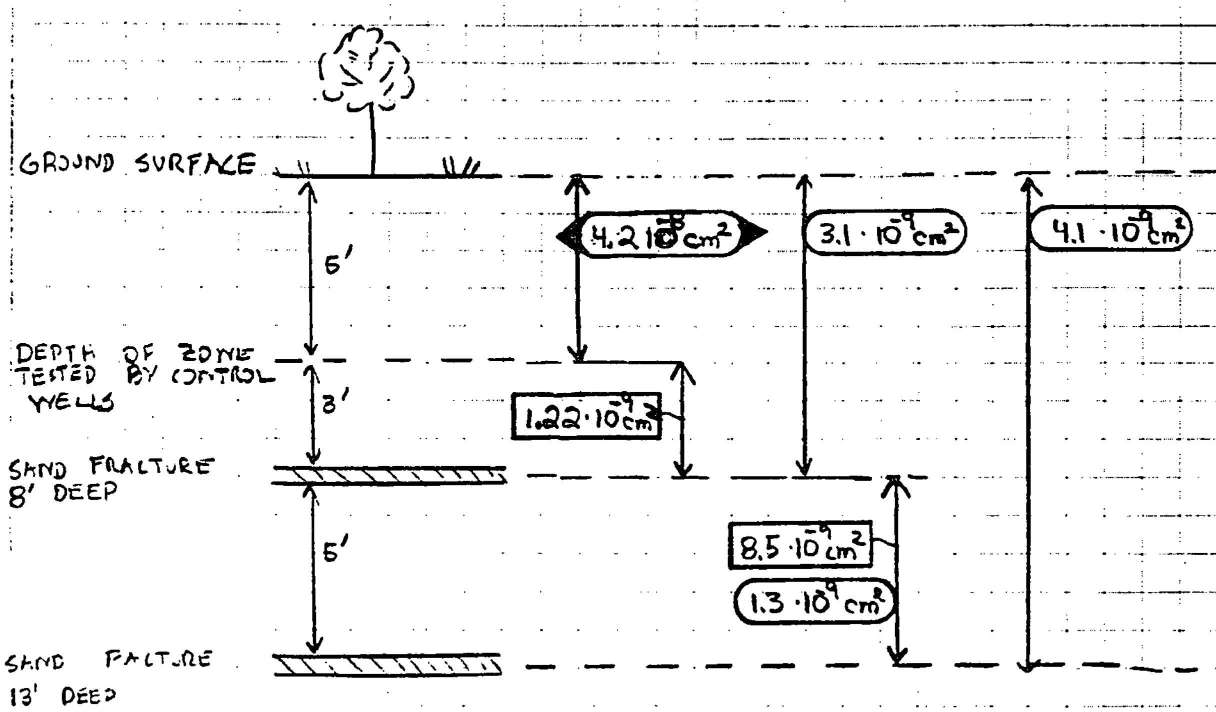
\# — The # is the permenbility estimated by the "Control Well" Test

\# — The # is the permeability estimated using the One-Dimensional Model and Field data for vacuums and flows across the soil zone indicated.

\# — The # is the permeability computed based on the estimated permeabilities. See Sheet 2 for more details.

Field Vertical Air Permeability Estimates at FW-B

FIGURE 5.2



NOTES

(#) The # is the vertical air permeability estimated from the results of the testing at the Control Well.

(#) The # is the vertical air permeability estimated using the One-Dimensional Model and the field data for vacuums and flows across the soil zone indicated.

[#] The # is the vertical air permeability computed based on the estimated permeabilities. The Computation is performed as follows:

L_1, K_1; L_2, K_2 → K_{eq}

$$K_{eq} = \frac{L_1 + L_2}{L_1/K_1 + L_2/K_2}$$

(Anderson & Woessner, 1992)

Typically K_{eq}, K_1, L_1, L_2 are known and K_2 is solved for.

was developed. The figures also include computed values of air permeability which are based on estimated permeabilities of two overlapping zones.

The results presented in these figures suggest the following:

- There is a general decreasing trend in vertical air permeability with increasing depth. The only exception to this is at FW-B where there is an increase in the estimated vertical air permeabilities as the depth increases from 8 feet to 13 feet below grade.

- The upper till deposits in the interval between the ground surface and a depth of five feet below grade have a vertical air permeability that is one to two orders of magnitude higher than the immediately underlying upper till deposits.

- An estimate of the representative field-determined vertical air permeability of the till at the Linemaster site can be made by taking the geometric mean of the vertical permeabilities shown in Figures 5.1 and 5.2 for a specific zone of soil. The representative vertical air permeabilities are summarized in the Table 5.4.

- The vertical permeabilities presented in Table 5.4 for the 18 to 28 foot zone suggest that the permeability of that zone is approximately one order of magnitude less than the zone above it. While this might suggest that the lower till zone, which starts at an approximate depth of 20 feet below grade in the area of the pilot test, is one order of magnitude less permeable than the upper till, consideration must be given to the other factors involved. During the field test, the 28-foot deep fracture was likely not entirely dewatered and the estimate of the vertical permeability assumed that it was. The moisture content of the lower till was also likely much higher in this zone. Based on these considerations, it is expected that during the full-scale operation of a remedial system, the vertical permeability of the lower till might be greater than indicated. It is conceivable that the vertical permeability of the lower till could be less than one order of magnitude less than the vertical permeability of the upper till.

5.3 Air Permeability Summary

A summary of the vertical and horizontal air permeability estimates for the till deposits at Linemaster are presented in Table 5.5. This table differentiates between the estimates based on laboratory analyses and estimates based on field observations that were discussed in the previous sections.

The vertical air permeability data summarized in Table 5.5 indicate a clear trend in decreasing permeability with increasing depth. The data also suggest that there is a two order of magnitude difference between the laboratory estimates and the field estimates of air permeability. This is consistent with the scale dependency of hydraulic conductivity measurements as discussed in Section 4.1. Additionally, from the ground surface to an approximate depth of five feet below grade, the bulk vertical air permeability is approximately one order of magnitude greater than the bulk horizontal air permeability and translates to an anisotropy ratio (k_v:k_h) of 28.

TABLE 5.4
REPRESENTATIVE FIELD MEASURED VERTICAL AIR PERMEABILITIES

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS
TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK CONNECTICUT
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Depth of Soil Zone	Estimated and Calculated Field Vertical Air Permeabilities (cm^2)	Representative Vertical Air Permeability (cm^2)
0 to 5 feet (Upper Till)	4.1×10^{-8} 4.3×10^{-8}	4.2×10^{-8}
5 to 8 feet (Upper Till)	1.2×10^{-9} 3.9×10^{-9}	2.2×10^{-9}
8 to 18 feet (Upper Till)	5.8×10^{-10} 9.0×10^{-10} 1.3×10^{-9} 8.5×10^{-9}	1.5×10^{-9}
18 to 28 feet (Lower Till)	1.6×10^{-10}	1.6×10^{-10}

TABLE 5.5
AIR PERMEABILITY SUMMARY TABLE

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS
TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK CONNECTICUT
FEBRUARY 1996

Depth of Soil Zone	Horizontal Air Permeability (cm^2)		Vertical Air Permeability (cm^2)	
	Laboratory	Field[1]	Laboratory[2]	Field[3]
0 to 5 feet (Upper Till)	N/A	1.5×10^{-9}	N/A	4.2×10^{-8}
5 to 8 feet (Upper Till)	N/A	N/A	N/A	2.2×10^{-9}
8 to 18 feet (Upper Till)	N/A	N/A	4.9×10^{-11}	1.5×10^{-9}
Greater than 20 feet (Lower Till)	N/A	N/A	6.3×10^{-12}	1.6×10^{-10}

Notes:
N/A- Data Not Available.
1 - The number presented in this column is the geometric mean of the field determined horizontal air permeabilities presented in Section 5.2.1
2 - The numbers presented in this column are the geometric mean of the laboratory determined vertical air permeabilities presented in Section 5.1.4
3- The number presented in this column is the geometric mean of the field determined vertical air permeabilities presented in Section 4.2.5 and Table 5.4.

6.0 DISCUSSION OF FINDINGS

Several conclusions are apparent based on the summary of the hydraulic conductivity and air permeability data presented and interpreted in Sections 4.0 and 5.0 of this Memorandum. This section of the Memorandum first presents these findings and then provides a preliminary discussion of the comparison of hydraulic conductivity and air permeability values.

6.1 General Conclusions

Based on the hydraulic conductivity values and air permeability values summarized in Tables 4.3 and 5.5, respectively, the following findings are presented:

1. Regardless of the scale of measurement (field or lab) or the orientation (horizontal or vertical), the hydraulic conductivity and air permeability of the till deposits in the vicinity of the manufacturing facility at Linemaster are observed to decrease with increasing depth. These data indicate that, in the Pilot Test area, the shallow portion of the upper till is the most permeable interval and the lower till unit (deeper than approximately 20 feet below grade) is the least permeable interval.

2. Field- and lab-derived hydraulic conductivity and air permeability values indicate that the vertical permeability of the upper till is up to one order of magnitude greater than the vertical permeability of the lower till.

3. Hydraulic conductivity data indicate that the upper till is approximately one half to one order of magnitude more permeable in the horizontal direction than the lower till.

4. Based on shallow, field air permeability testing results, the uppermost five feet of the upper till is anisotropic. Within this interval, the vertical air permeability in the bulk till is approximately one order of magnitude greater than the horizontal air permeability of the bulk till.

5. The effects of scale dependency upon vertical air permeability measurements are apparent. For similar depth intervals, in-situ testing estimates of the vertical air permeability of the bulk till are approximately one to two orders of magnitude greater than the laboratory till matrix vertical air permeability estimates. This increase supports the conclusion that naturally occurring fractures are present within the Linemaster till deposits and are responsible for substantial secondary porosity.

6. In consideration of the scale dependency of hydraulic conductivity and air permeability measurements, the flow rates of groundwater and air extracted from the bulk till are most representatively estimated using bulk permeability values

derived from in-situ testing methods, whereas, water and vapor transport through the till matrix are most accurately evaluated using laboratory-derived estimates of the matrix permeability.

6.2 Comparison of Hydraulic Conductivity and Air Permeability Values

The hydraulic conductivity and air permeability of soil are functions of the intrinsic permeability. Therefore, the hydraulic conductivities discussed in Section 4.0 and the air permeabilities discussed in Section 5.0 theoretically can be directly compared. The theoretical relationship (Freeze & Cherry, 1979) between the intrinsic permeability k_i and hydraulic conductivity K is given by:

$$K = k_i \frac{\mu}{g\rho}$$

where μ is the viscosity of water, ρ is the density of water, and g is the acceleration due to gravity. The theoretical relationship (DiGiulio, 1992) between the intrinsic permeability k_i and air permeability k_a is given by:

$$k_a = k_i k_r$$

where k_r, the relative permeability, represents the fraction of the total intrinsic air permeability that is available for air flow. The relative permeability ranges from 0 to 1.0 and is dependent upon the moisture content of the soil. Very moist soil would have a k_r close to zero and a very dry soil would have a k_r close to one.

The above theoretical relationships assume darcian flow through a porous granular medium. These relationships also assume that the physical properties of the soil remain constant (i.e., are not influenced by shrinkage or expansion).

The vertical and horizontal estimates of hydraulic conductivity and air permeability associated with different subsurface intervals at Linemaster are presented in Tables 6.1 and 6.2, respectively. The data in each of these two tables have been distinguished on the basis and applicability of the method of parameter measurement. As discussed previously in Section 4.1, the laboratory hydraulic conductivity and air permeability data are representative of the till matrix. Due to the sample orientation during testing in all cases, these data are considered estimates of the vertical hydraulic conductivity or vertical air permeability. The field results are believed to be representative of the permeabilities of the till matrix.

Also included in Tables 6.1 and 6.2 are calculated intrinsic permeability values, which were derived from the estimated hydraulic conductivity values; relative permeability values, which were calculated based on the calculated intrinsic permeabilities and the estimated air permeability values, also are included in these tables.

In the cases where estimates of both the air permeability and hydraulic conductivity exist for

TABLE 6.2
HORIZONTAL HYDRAULIC CONDUCTIVITY, INTRINSIC PERMEABILITY AND AIR PERMEABILITY SUMMARY TABLE

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
FEBRUARY 1996

TILL MATRIX

SOIL ZONE AND DEPTH	ESTIMATED HORIZONTAL HYDRAULIC CONDUCTIVITY K (FEET/DAY)	CALCULATED HORIZONTAL INTRINSIC PERMEABILITY ki (*1) (CM^2)	ESTIMATED HORIZONTAL AIR PERMEABILITY ka (CM^2)	CALCULATED RELATIVE PERMEABILITY kr (*2) (-)
UPPER TILL				
Surface to 5 feet	N/A	N/A	N/A	N/A
5 feet to 8 feet	N/A	N/A	N/A	N/A
8 feet to 18 feet	N/A	N/A	N/A	N/A
LOWER TILL				
Greater than 20 feet	N/A	N/A	N/A	N/A

BULK TILL

SOIL ZONE AND DEPTH	ESTIMATED HORIZONTAL HYDRAULIC CONDUCTIVITY K (*3) (FEET/DAY)	CALCULATED HORIZONTAL INTRINSIC PERMEABILITY ki (*1) (CM^2)	ESTIMATED HORIZONTAL AIR PERMEABILITY ka (*4) (CM^2)	CALCULATED RELATIVE PERMEABILITY kr (*2) (-)
UPPER TILL				
Surface to 5 feet	N/A	N/A	1.5E-09	N/A
5 feet to 8 feet	N/A	N/A	N/A	N/A
8 feet to 18 feet	1.5E-02	7.1E-11	N/A	N/A
LOWER TILL				
Greater than 20 feet	3.0E-03	1.4E-11	N/A	N/A

NOTES
N/A - Data not available
(*1) - Intrinsic permeabilities were obtained from the hydraulic conductivity values given by using the formula $ki = K * u / (ro * g)$ where u and ro are the viscosity and density of water respectively. g is the gravitational constant. For site conditions and units given, ki = K*4.71E-9.
(*2) - The calculated relative permeability was computed by kr = ka/ki. In theory, kr represents the fraction of the intrinsic permeability that is available for air flow and $0 < kr < 1$.
(*3) - Hydraulic Conductivity estimates of the bulk till were obtained from slug tests.
(*4) - Air permeability estimates of the bulk till were obtained from field tests.

the same depth interval and the values were estimated from tests conducted at the same relative scale (lab or field), it is possible to directly compare the results via the relative permeability. As discussed previously in this section, the relative permeability theoretically should range between 1.0 and 0; however, the field estimates indicate that the relative permeability actually ranges from 25 to 124. Theoretically, this is not possible. To explain this phenomenon, the theory and assumptions must be reconsidered. Preliminary causes and explanations currently under consideration include the following:

- Because field and lab permeability measurements could not be made at the same location in the subsurface soils, the observed variance may due to the heterogeneity and anisotropy of the till deposits.

- The presence of naturally-occurring fractures may govern the air flow through the soil. Since these fractures are likely planar in nature, the flow through them likely is not adequately represented by theory developed for porous media.

- The flow of air and groundwater may be influenced differently by the presence of fractures, whether they be of natural origin or were created hydraulically during the Pilot Test.

- The glacial tills present at Linemaster are poorly-sorted heterogeneous deposits with silt- and clay-sized particles constituting the predominant grain size. It is likely that clay minerals account for a significant portion of the fine-grained particles. It is possible that as the till was dewatered during the Pilot Test, shrinkage occurred and resulted in increased secondary porosity, which in turn caused an increase the air permeability.

7.0 REFERENCES

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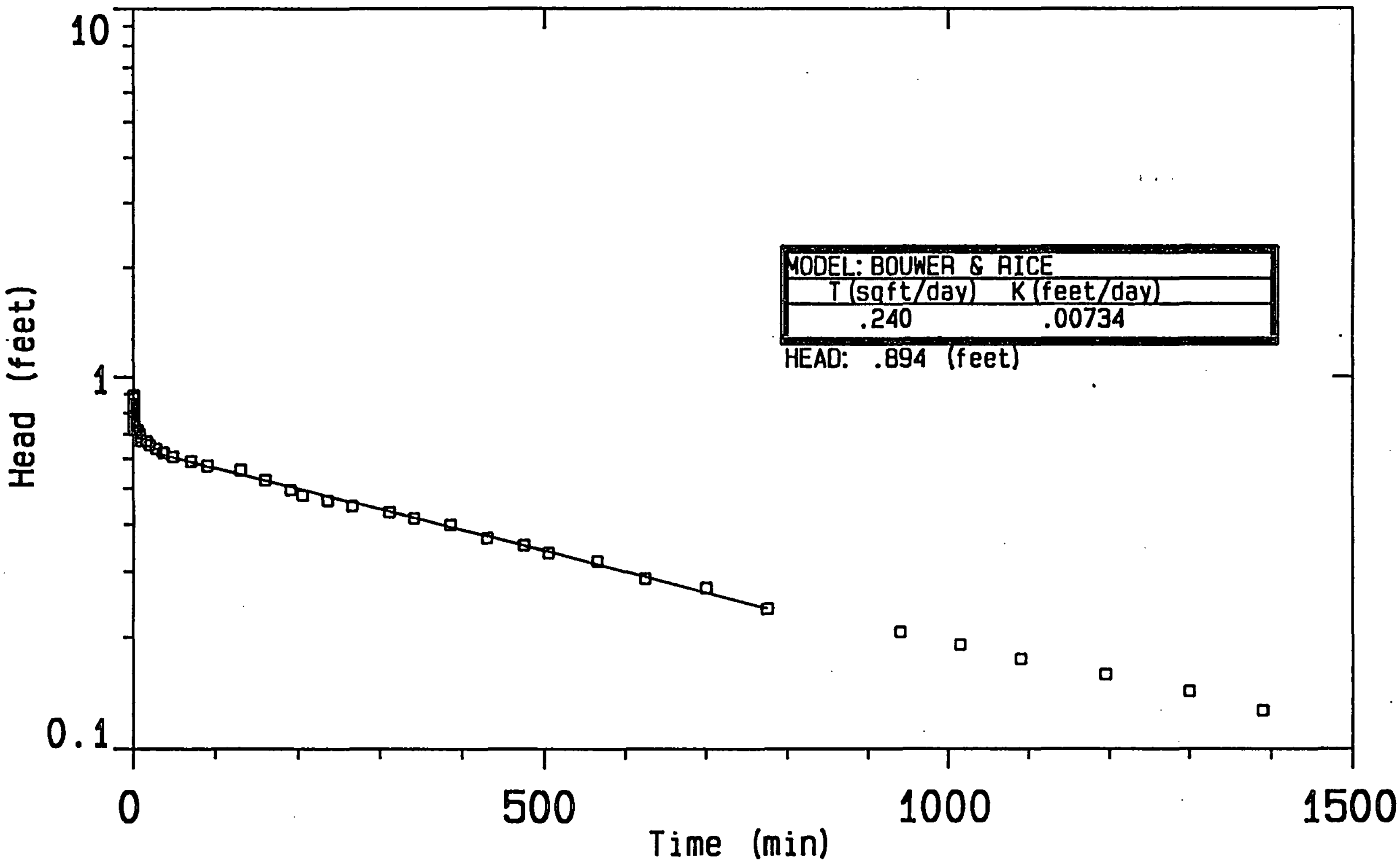
FOR DRAWINGS, PLEASE SEE BOUND

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS TECHNICAL MEMORANDUM

DATED FEBRUARY 1996

APPENDIX A
SLUG TEST ANALYTICAL DATA

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS
TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK CONNECTICUT
FEBRUARY 1996



for: LINEMASTER SWITCH CORPORATION	WELL SLUG TEST, C1-PZ-22
by: Fuss & O'Neill Inc.	LINEMASTER SWITCH CORPORATION
Aquifer: Overburden Thickness: 32.7 Depth: 22.6 feet Screen: Top: 22.0 Base: 22.5 feet Effective Diameter .380 feet	WOODSTOCK, CT Date: OCT 25 95 Well No.: C1P22

DATA SET: C1PZ22SL

CLIENT:	LINEMASTER SWITCH CORPORATION	DATE:	OCT 25 95
LOCATION:	LINEMASTER SWITCH CORPORATION	WELL NO.:	C1P22
COUNTY:	WOODSTOCK, CT	INIT. HEAD:	0.89 feet
PROJECT:	WELL SLUG TEST, C1-PZ-22	WELL DEPTH:	22.67 feet
AQUIFER:	Overburden	THICKNESS:	32.70 feet
DEPTH TO WATER IN WELL:	13.80 feet	DURATION OF TEST :	1390.00 min
BOREHOLE DIA.:	0.730 feet	CASING DIAMETER:	0.125 feet
SCREEN DIAMETER:	0.125 feet	EFFECTIVE DIAMETER:	0.381 feet
DEPTH TO AQUIFER:	13.800 feet	PACKING POROSITY:	25.000 %

WELL SCREENED FROM 22.00 TO 22.50 feet

All depths are from Surface

FITTING ERROR: 1.761 PERCENT

UNCONFINED PARTIALLY PENETRATED AQUIFER (Bouwer & Rice)

MODEL PARAMETERS:

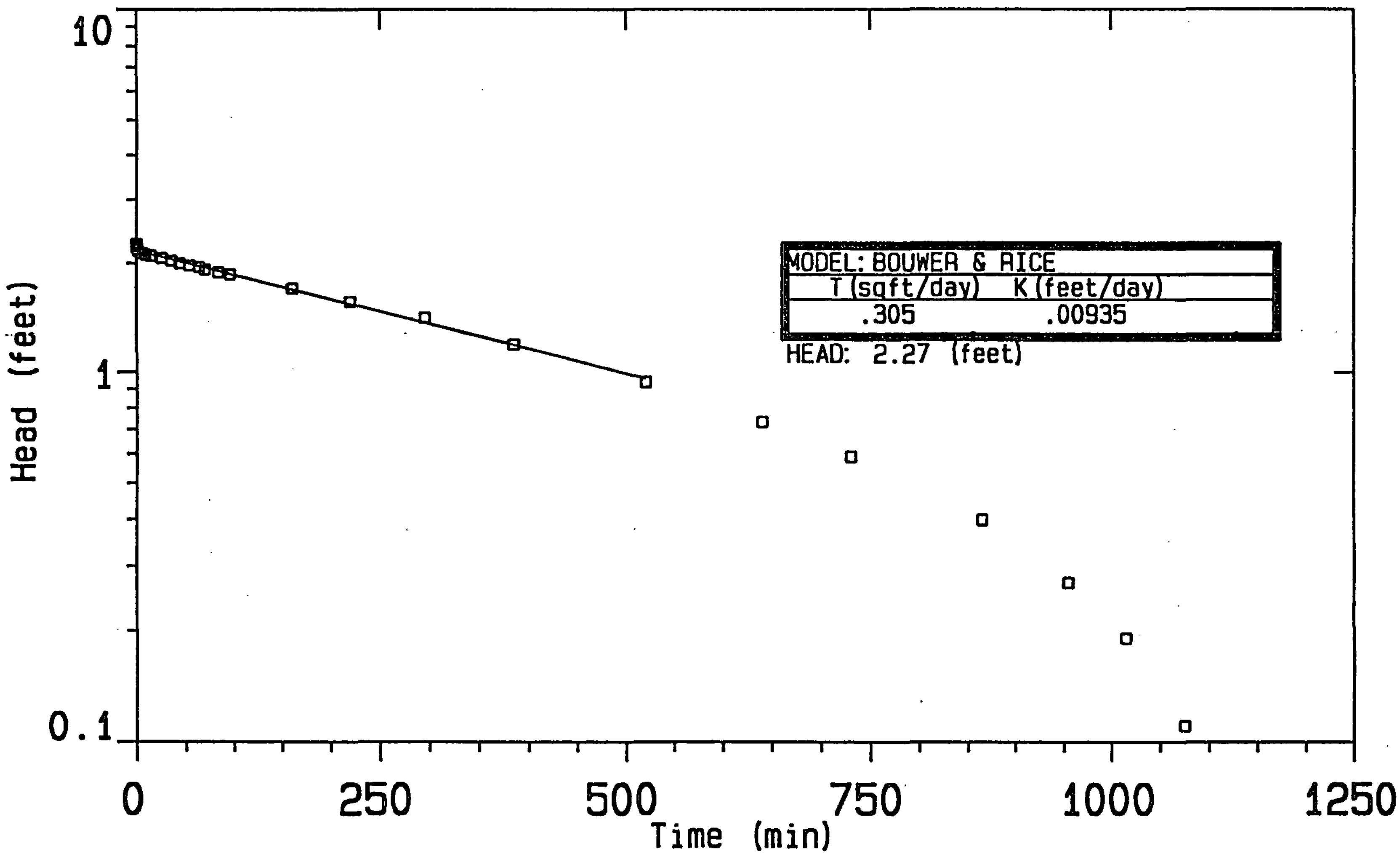
TRANSM: 2.40E-01 sqft/day FREE

COND: 7.34E-03 feet/day FREE

No.	TIME (min)	Head, H (feet) DATA	SYNTHETIC	DIFFERENCE (percent)
1	0.0166	0.894		
2	0.0233	0.830		
3	0.0300	0.814		
4	0.0400	0.814		
5	0.0466	0.798		
6	0.0533	0.830		
7	0.0600	0.734		
8	0.0733	0.878		
9	0.0766	0.846		
10	0.0900	0.831		
11	0.0966	0.814		
12	0.100	0.798		
13	0.140	0.782		
14	0.200	0.782		
15	0.260	0.782		
16	0.300	0.782		
17	0.466	0.766		

No.	TIME (min)	Head, H (feet) DATA	SYNTHETIC	DIFFERENCE (percent)
18	0.600	0.750		
19	0.800	0.750		
20	0.900	0.750		
21	1.20	0.718		
22	3.00	0.718		
23	5.00	0.718		
24	7.00	0.702		
25	8.00	0.686		
26	9.20	0.670		
27	16.00	0.670		
28	20.00	0.654		
29	28.00	0.639	0.620	2.87
30	36.00	0.623	0.614	1.38
31	48.00	0.607	0.605	0.328
32	70.00	0.591	0.588	0.464
33	90.00	0.575	0.573	0.272
34	130.0	0.559	0.544	2.52
35	160.0	0.527	0.524	0.487
36	190.0	0.495	0.504	-1.96
37	205.0	0.479	0.495	-3.37
38	235.0	0.463	0.476	-2.92
39	265.0	0.447	0.458	-2.60
40	310.0	0.431	0.433	-0.479
41	340.0	0.415	0.416	-0.433
42	385.0	0.399	0.393	1.36
43	430.0	0.367	0.371	-1.24
44	475.0	0.351	0.350	0.0460
45	505.0	0.335	0.337	-0.794
46	565.0	0.319	0.312	1.95
47	625.0	0.287	0.289	-0.946
48	700.0	0.271	0.263	2.85
49	775.0	0.239	0.239	-0.101
50	940.0	0.207		
51	1015.0	0.191		
52	1090.0	0.175		
53	1195.0	0.159		
54	1300.0	0.143		
55	1390.0	0.127		

CURRENT RESOLUTION MATRIIX NOT AVAILABLE



for: LINEMASTER SWITCH CORPORATION	WELL SLUG TEST, C1-PZ-44
by: Fuss & O'Neill Inc.	LINEMASTER SWITCH CORPORATION
Aquifer: Overburden Thickness: 32.7 Depth: 45.2 feet Screen: Top: 44.0 Base: 44.5 feet Effective Diameter .380 feet	WOODSTOCK, CT
	Date: OCT 25 95 | Well No.: C1P44

DATA SET: C1PZ44SL

CLIENT:	LINEMASTER SWITCH CORPORATION	DATE:	OCT 25 95
LOCATION:	LINEMASTER SWITCH CORPORATION	WELL NO.:	C1P44
COUNTY:	WOODSTOCK, CT	INIT. HEAD:	2.27 feet
PROJECT:	WELL SLUG TEST, C1-PZ-44	WELL DEPTH:	45.19 feet
AQUIFER:	Overburden	THICKNESS:	32.70 feet
DEPTH TO WATER IN WELL:	27.03 feet	DURATION OF TEST :	1075.00 min
BOREHOLE DIA.:	0.730 feet	CASING DIAMETER:	0.125 feet
SCREEN DIAMETER:	0.125 feet	EFFECTIVE DIAMETER:	0.381 feet
DEPTH TO AQUIFER:	13.800 feet	PACKING POROSITY:	25.000 %

WELL SCREENED FROM 44.00 TO 44.50 feet

All depths are from Surface

FITTING ERROR: 1.391 PERCENT

UNCONFINED PARTIALLY PENETRATED AQUIFER (Bouwer & Rice)

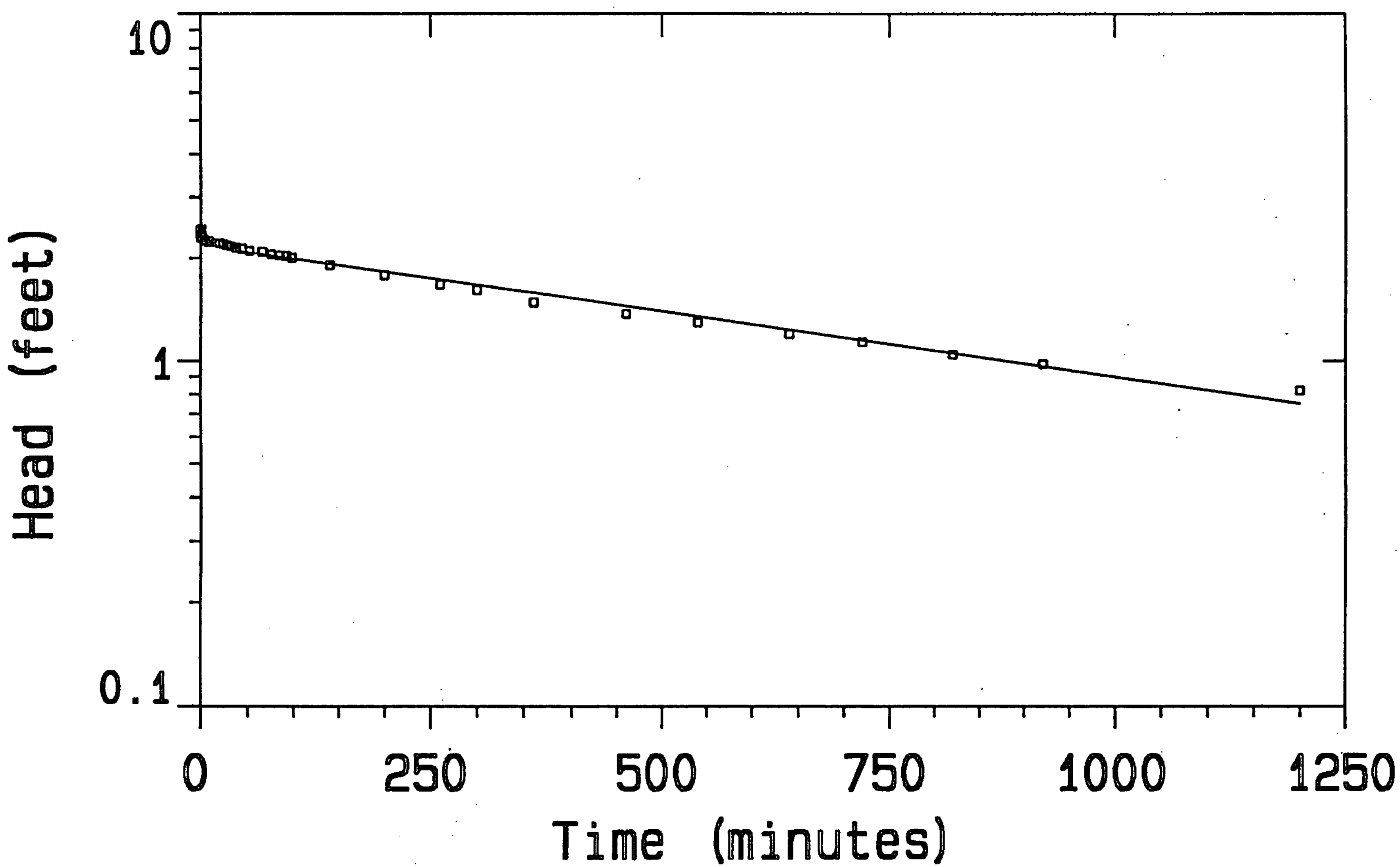
MODEL PARAMETERS:

TRANSM: 3.06E-01 sqft/day FREE   COND: 9.35E-03 feet/day FREE

No.	TIME (min)	Head, H (feet) DATA	SYNTHETIC	DIFFERENCE (percent)
1	0.206	2.27		
2	0.216	2.26		
3	0.223	2.24		
4	0.236	2.22		
5	0.250	2.21	2.16	1.88
6	0.293	2.22	2.16	2.33
7	0.566	2.21	2.16	1.93
8	0.950	2.19	2.16	1.09
9	1.80	2.16	2.16	-0.143
10	3.40	2.14	2.15	-0.826
11	6.00	2.13	2.14	-0.887
12	9.20	2.11	2.13	-1.33
13	14.00	2.10	2.12	-1.05
14	26.00	2.07	2.08	-0.605
15	36.00	2.03	2.05	-0.991
16	44.00	2.00	2.02	-1.22
17	54.00	1.97	1.99	-1.16

No.	TIME (min)	Head, H (feet) DATA	SYNTHETIC	DIFFERENCE (percent)
18	64.00	1.95	1.96	-0.616
19	70.00	1.92	1.94	-1.23
20	84.00	1.89	1.90	-0.604
21	96.00	1.86	1.86	-0.321
22	160.0	1.70	1.68	0.720
23	220.0	1.56	1.53	1.52
24	295.0	1.41	1.36	3.14
25	385.0	1.19	1.18	0.348
26	520.0	0.939	0.959	-2.18
27	640.0	0.732		
28	730.0	0.589		
29	865.0	0.398		
30	955.0	0.270		
31	1015.0	0.191		
32	1075.0	0.111		

CURRENT RESOLUTION MATRIIX NOT AVAILABLE



MODEL TYPE: BOUWER and RICE	
CONDUCTIVITY: .001168 ft/day	
TRANSMISSIVITY: .01848 sq. ft/day	
INITIAL HEAD: 2.320 ft	
Data Set: MW31T2	Date: 30-OCT-95

for: LINEMASTER SWITCH CORP.
by: Fuss & O'Neill, Inc.

WELL DATA: Units: ft
AQUIFER: UNCONFINED
THICKNESS: 15.82
SCREEN: top: 20.00 base: 30.00
DIAMETER: casing: .1660 intake: .6660
DEPTH: Water Table: 16.36 TD: 32.18

Well Slug Test Data

Well: MW-31T

LINEMASTER SWITCH CORP.
WOODSTOCK, CONNECTICUT

DATA SET: MW31T2

CLIENT:	LINEMASTER SWITCH CORP.	DATE:	30-OCT-95
LOCATION:	LINEMASTER SWITCH CORP.	WELL NO.:	MW-31T
COUNTY:	WOODSTOCK, CONNECTICUT	WELL DEPTH:	32.18 ft
PROJECT:	Well Slug Test Data	WATER TABLE:	16.360 ft
AQUIFER:	UNCONFINED	THICKNESS:	15.82 ft
INTAKE RADIUS:	0.333 ft	CASING RADIUS:	0.083 ft
SCREEN TOP:	20.000 ft	SCREEN BASE:	30.00 ft
INITIAL HEAD:	2.320 ft	TRANS. RATIO:	1.0000

MODEL PARAMETERS:

TRANSMISSIVITY: 0.01848square ft/day

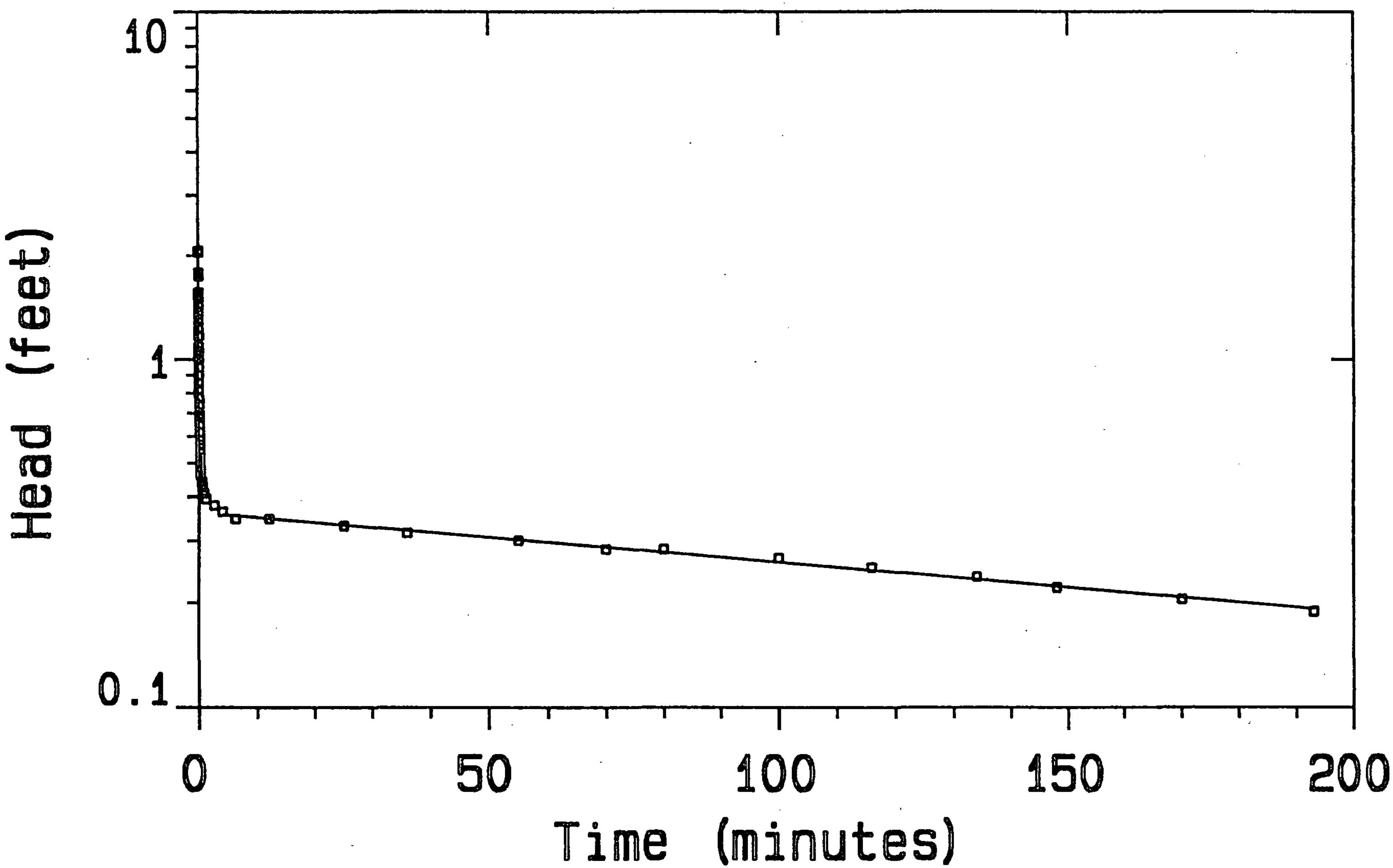
CONDUCTIVITY: 0.00117 ft/day

MODEL TYPE: UNCONFINED PARTIALLY PENETRATED AQUIFER (Bouwer & Rice)

No.	TIME (mins)	Head, H (ft) DATA	SYNTHETIC	DIFFERENCE (percent)
1	0.0566	2.42		
2	0.0600	2.41		
3	0.0800	2.37		
4	0.0900	2.37		
5	0.116	2.33		
6	0.126	2.32		
7	0.160	2.30		
8	0.196	2.30		
9	0.273	2.29		
10	0.700	2.30		
11	1.60	2.29		
12	2.40	2.27		
13	4.00	2.26		
14	5.80	2.24		
15	7.80	2.22		
16	8.60	2.24		
17	9.40	2.22		
18	20.00	2.21	2.14	3.19
19	24.00	2.19	2.13	2.87
20	28.00	2.18	2.12	2.50
21	32.00	2.16	2.11	2.13
22	38.00	2.14	2.10	1.93
23	44.00	2.13	2.09	1.72

No.	TIME (mins)	Head, H (ft) DATA	SYNTHETIC	DIFFERENCE (percent)
24	52.00	2.10	2.08	0.976
25	66.00	2.08	2.05	1.44
26	76.00	2.05	2.03	0.797
27	84.00	2.03	2.02	0.725
28	92.00	2.03	2.00	1.13
29	98.00	2.00	1.99	0.436
30	140.0	1.91	1.92	-0.687
31	200.0	1.78	1.82	-2.20
32	260.0	1.67	1.73	-3.32
33	300.0	1.61	1.67	-3.62
34	360.0	1.48	1.58	-6.65
35	460.0	1.37	1.44	-5.40
36	540.0	1.29	1.35	-4.17
37	640.0	1.20	1.23	-2.86
38	720.0	1.13	1.15	-1.21
39	820.0	1.04	1.05	-0.969
40	920.0	0.979	0.963	1.56
41	1200.0	0.821	0.751	8.43

CURRENT RESOLUTION MATRIIX NOT AVAILABLE



MODEL TYPE: BOUWER and RICE	for: LINEMASTER SWITCH CORP.	Well Slug Test Data
CONDUCTIVITY: .01365 ft/day	by: Fuss & O'Neill, Inc.	Well: MW-32T
TRANSMISSIVITY: .1411 sq. ft/day	WELL DATA: Units: ft	LINEMASTER SWITCH CORP.
INITIAL HEAD: 2.100 ft	AQUIFER: UNCONFINED	WOODSTOCK, CONNECTICUT
Data Set: MW32T	THICKNESS: 10.34	
Date: 13-MAR-63	SCREEN: top: 16.00 base: 26.00	
	DIAMETER: casing: .1666 intake: .6860	
	DEPTH: Water Table: 17.96 TD: 28.30	

DATA SET: MW32T

CLIENT:	LINEMASTER SWITCH CORP.	DATE:	13-MAR-63
LOCATION:	LINEMASTER SWITCH CORP.	WELL NO.:	MW-32T
COUNTY:	WOODSTOCK, CONNECTICUT	WELL DEPTH:	28.30 ft
PROJECT:	Well Slug Test Data	WATER TABLE:	17.960 ft
AQUIFER:	UNCONFINED	THICKNESS:	10.34 ft
INTAKE RADIUS:	0.343 ft	CASING RADIUS:	0.083 ft
SCREEN TOP:	16.000 ft	SCREEN BASE:	26.00 ft
INITIAL HEAD:	2.100 ft	TRANS. RATIO:	1.0000

MODEL PARAMETERS:

TRANSMISSIVITY: 0.14116square ft/day

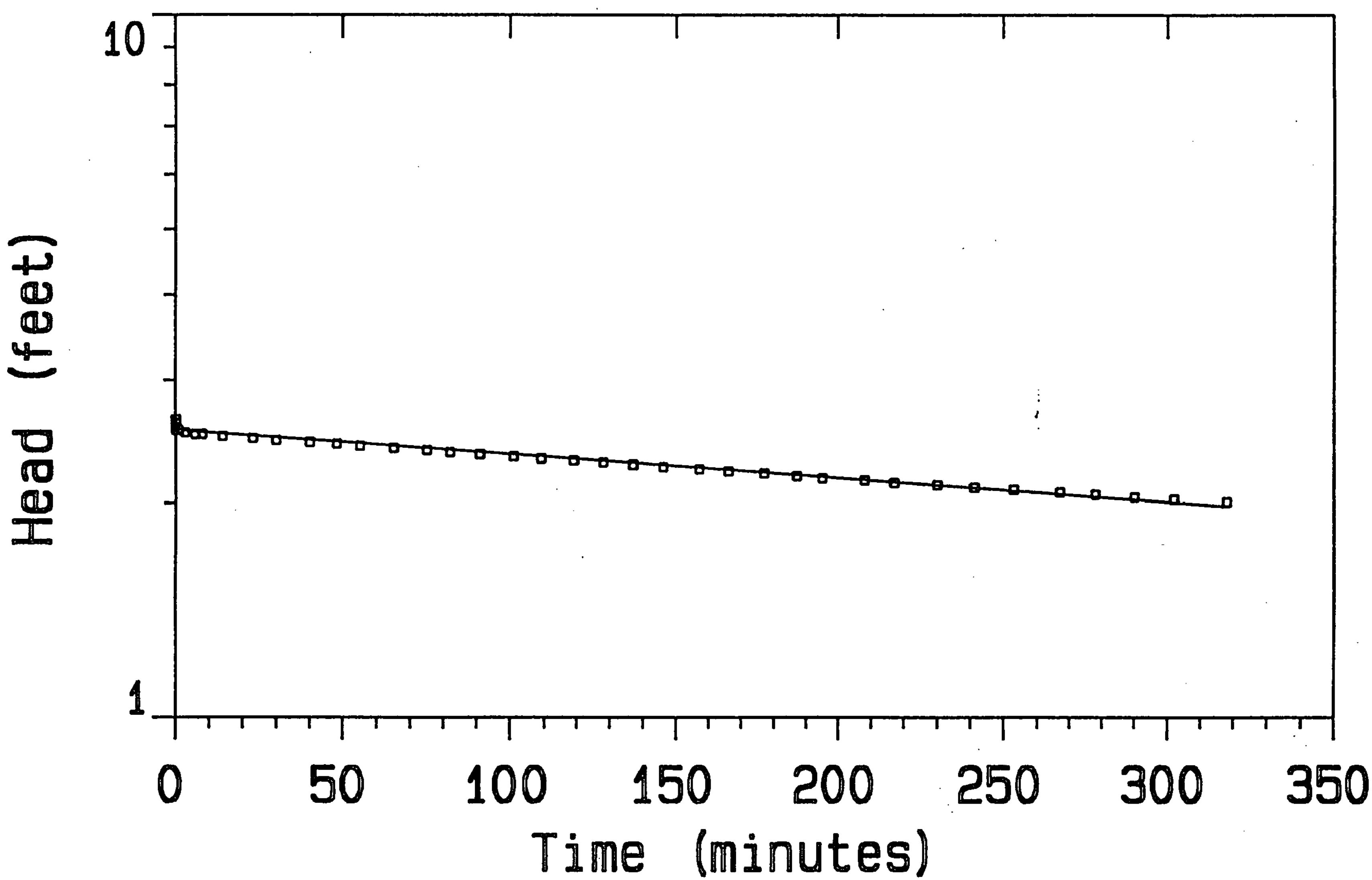
CONDUCTIVITY: 0.01365 ft/day

MODEL TYPE: UNCONFINED PARTIALLY PENETRATED AQUIFER (Bouwer & Rice)

No.	TIME (mins)	Head, H (ft) DATA	SYNTHETIC	DIFFERENCE (percent)
1	0.00660	2.07		
2	0.0100	2.03		
3	0.0133	1.53		
4	0.0166	1.50		
5	0.0200	1.78		
6	0.0233	1.73		
7	0.0266	1.50		
8	0.0333	1.56		
9	0.0400	1.42		
10	0.0500	1.37		
11	0.0533	1.31		
12	0.0566	1.28		
13	0.0633	1.23		
14	0.0700	1.16		
15	0.0800	1.09		
16	0.0833	1.05		
17	0.0866	1.02		
18	0.0900	1.01		
19	0.0933	0.995		
20	0.100	0.948		
21	0.106	0.900		
22	0.113	0.853		
23	0.120	0.821		

No.	TIME (mins)	Head, H (ft) DATA	SYNTHETIC	DIFFERENCE (percent)
24	0.126	0.774		
25	0.133	0.742		
26	0.143	0.695		
27	0.150	0.679		
28	0.160	0.647		
29	0.166	0.616		
30	0.180	0.584		
31	0.193	0.568		
32	0.203	0.537		
33	0.220	0.521		
34	0.233	0.505		
35	0.246	0.489		
36	0.266	0.474		
37	0.300	0.458		
38	0.466	0.442		
39	0.600	0.426		
40	0.800	0.410		
41	1.20	0.395		
42	2.60	0.379		
43	4.00	0.363	0.356	1.75
44	6.20	0.347	0.354	-2.03
45	12.00	0.347	0.347	-0.122
46	25.00	0.331	0.332	-0.600
47	36.00	0.316	0.321	-1.65
48	55.00	0.300	0.301	-0.639
49	70.00	0.284	0.287	-1.22
50	80.00	0.284	0.278	2.02
51	100.0	0.268	0.260	2.73
52	116.0	0.252	0.247	1.82
53	134.0	0.237	0.233	1.57
54	148.0	0.221	0.222	-0.837
55	170.0	0.205	0.207	-1.17
56	193.0	0.189	0.192	-1.79

CURRENT RESOLUTION MATRIIX NOT AVAILABLE



MODEL TYPE: BOUWER and RICE

CONDUCTIVITY: .001120 ft/day

TRANSMISSIVITY: .01524 sq. ft/day

INITIAL HEAD: 2.500 ft

Data Set: MW32SB

Date: 31-OCT-95

for: LINEMASTER SWITCH CORP.

by: Fuss & O'Neill, Inc.

WELL DATA: Units: ft

AQUIFER: UNCONFINED

THICKNESS: 13.60

SCREEN: top: 74.00 base: 84.00

DIAMETER: casing: .1660 intake: .2500

DEPTH: Water Table: 73.40 TD: 87.00

Well Slug Test Data

Well: MW-32SB

LINEMASTER SWITCH CORP.

WOODSTOCK, CONNECTICUT

DATA SET: MW32SB

CLIENT:	LINEMASTER SWITCH CORP.	DATE:	31-OCT-95
LOCATION:	LINEMASTER SWITCH CORP.	WELL NO.:	MW-32SB
COUNTY:	WOODSTOCK, CONNECTICUT	WELL DEPTH:	87.00 ft
PROJECT:	Well Slug Test Data	WATER TABLE:	73.400 ft
AQUIFER:	UNCONFINED	THICKNESS:	13.60 ft
INTAKE RADIUS:	0.125 ft	CASING RADIUS:	0.083 ft
SCREEN TOP:	74.000 ft	SCREEN BASE:	84.00 ft
INITIAL HEAD:	2.500 ft	TRANS. RATIO:	1.0000

MODEL PARAMETERS:

TRANSMISSIVITY: 0.01524square ft/day

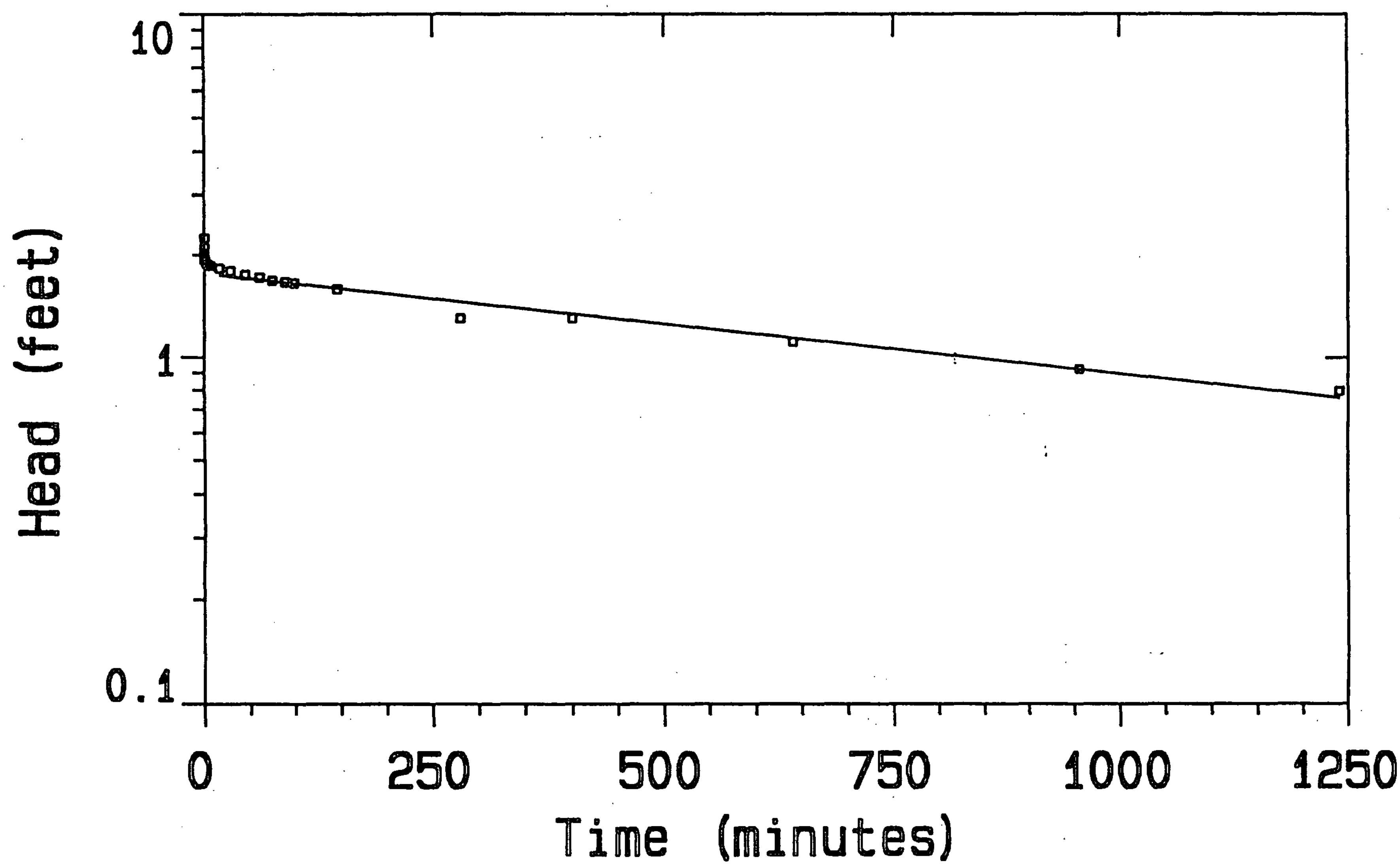
CONDUCTIVITY: 0.00112 ft/day

MODEL TYPE: UNCONFINED PARTIALLY PENETRATED AQUIFER (Bouwer & Rice)

No.	TIME (mins)	Head, H (ft) DATA	SYNTHETIC	DIFFERENCE (percent)
1	0.0600	2.63	2.54	3.57
2	0.0800	2.60	2.54	2.39
3	0.100	2.58	2.54	1.79
4	0.130	2.60	2.54	2.39
5	0.160	2.55	2.54	0.568
6	0.180	2.54	2.54	-0.0560
7	0.250	2.55	2.54	0.575
8	0.300	2.55	2.54	0.579
9	0.600	2.55	2.54	0.602
10	0.900	2.54	2.54	1.877E-04
11	1.00	2.54	2.53	0.00800
12	2.80	2.52	2.53	-0.484
13	6.00	2.50	2.52	-0.873
14	8.00	2.50	2.52	-0.715
15	14.00	2.49	2.51	-0.888
16	23.00	2.47	2.49	-0.829
17	30.00	2.46	2.48	-0.931
18	40.00	2.44	2.46	-0.802
19	48.00	2.42	2.44	-0.834
20	55.00	2.41	2.43	-0.949
21	65.00	2.39	2.41	-0.833
22	75.00	2.38	2.39	-0.721
23	82.00	2.36	2.38	-0.850

No.	TIME (mins)	Head, H (ft) DATA	SYNTHETIC	DIFFERENCE (percent)
24	91.00	2.34	2.36	-0.826
25	101.0	2.33	2.34	-0.728
26	109.0	2.31	2.33	-0.792
27	119.0	2.30	2.31	-0.704
28	128.0	2.28	2.29	-0.699
29	137.0	2.26	2.28	-0.699
30	146.0	2.25	2.26	-0.704
31	157.0	2.23	2.24	-0.557
32	166.0	2.22	2.23	-0.572
33	177.0	2.20	2.21	-0.436
34	187.0	2.18	2.19	-0.383
35	195.0	2.17	2.18	-0.493
36	208.0	2.15	2.16	-0.216
37	217.0	2.14	2.14	-0.258
38	230.0	2.12	2.12	0.00676
39	241.0	2.10	2.10	0.109
40	253.0	2.09	2.08	0.284
41	267.0	2.07	2.06	0.608
42	278.0	2.06	2.04	0.693
43	290.0	2.04	2.02	0.849
44	302.0	2.02	2.00	1.04
45	318.0	2.01	1.98	1.50

CURRENT RESOLUTION MATRIIX NOT AVAILABLE



MODEL TYPE: BOUWER and RICE	for: LINEMASTER SWITCH CORP.	Well Slug Test Data
CONDUCTIVITY: .001375 ft/day	by: Fuss & O'Neill, Inc.	Well: MW-33T
TRANSMISSIVITY: .02935 sq. ft/day	WELL DATA: Units: ft	LINEMASTER SWITCH CORP.
INITIAL HEAD: 2.500 ft	AQUIFER: UNCONFINED	WOODSTOCK, CONNECTICUT
Data Set: MW33T	THICKNESS: 21.34	
Date: 25-OCT-95	SCREEN: top: 25.00 base: 35.00	
	DIAMETER: casing: .1666 intake: .6860	
	DEPTH: Water Table: 14.02 TD: 35.36	

DATA SET: MW33T

CLIENT:	LINEMASTER SWITCH CORP.	DATE:	25-OCT-95
LOCATION:	LINEMASTER SWITCH CORP.	WELL NO.:	MW-33T
COUNTY:	WOODSTOCK, CONNECTICUT	WELL DEPTH:	35.36 ft
PROJECT:	Well Slug Test Data	WATER TABLE:	14.020 ft
AQUIFER:	UNCONFINED	THICKNESS:	21.34 ft
INTAKE RADIUS:	0.343 ft	CASING RADIUS:	0.083 ft
SCREEN TOP:	25.000 ft	SCREEN BASE:	35.00 ft
INITIAL HEAD:	2.500 ft	TRANS. RATIO:	1.0000

MODEL PARAMETERS:

TRANSMISSIVITY: 0.02935square ft/day

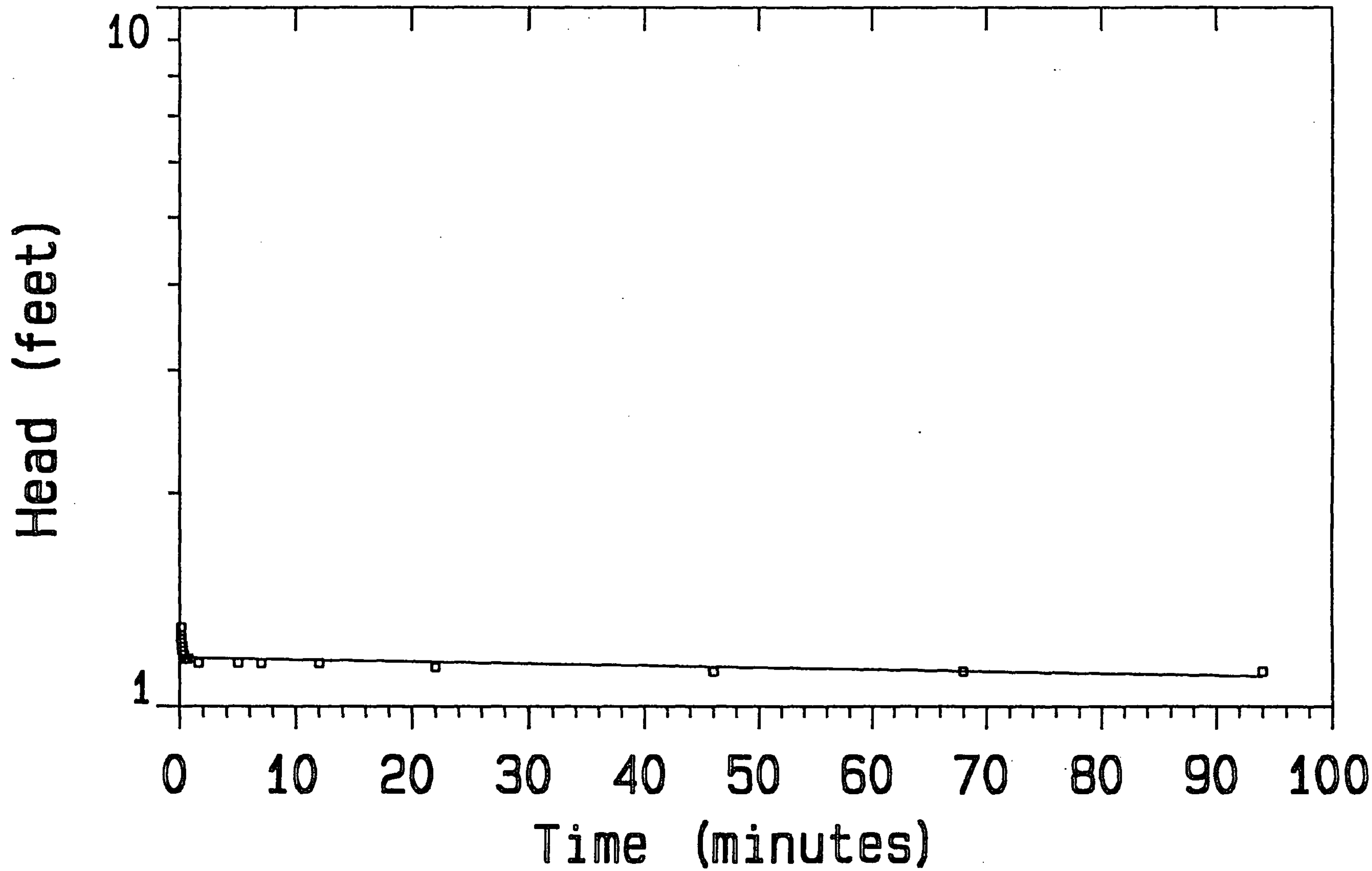
CONDUCTIVITY: 0.00138 ft/day

MODEL TYPE: UNCONFINED PARTIALLY PENETRATED AQUIFER (Bouwer & Rice)

No.	TIME (mins)	Head, H (ft) DATA	SYNTHETIC	DIFFERENCE (percent)
1	0.106	2.24		
2	0.113	2.10		
3	0.146	2.02		
4	0.170	2.00		
5	0.183	1.97		
6	0.233	1.95		
7	0.326	1.94		
8	0.766	1.92		
9	2.20	1.89		
10	4.20	1.87		
11	7.00	1.86		
12	16.00	1.83	1.74	4.53
13	28.00	1.79	1.73	3.62
14	44.00	1.75	1.71	2.10
15	60.00	1.71	1.69	1.35
16	74.00	1.68	1.67	0.440
17	88.00	1.67	1.66	0.439
18	98.00	1.65	1.65	0.157
19	145.0	1.59	1.60	-0.530
20	280.0	1.30	1.46	-11.88
21	400.0	1.30	1.34	-3.12
22	640.0	1.11	1.14	-2.62
23	955.0	0.923	0.922	0.00563

No.	TIME (mins)	Head, H (ft) DATA	SYNTHETIC	DIFFERENCE (percent)
24	1240.0	0.796	0.760	4.46

CURRENT RESOLUTION MATRIIX NOT AVAILABLE



MODEL TYPE: BOUWER and RICE	
CONDUCTIVITY: .002807 ft/day	
TRANSMISSIVITY: .1373 sq. ft/day	
INITIAL HEAD: 1.300 ft	
Data Set: MW34T	Date: 31-OCT-95

for: LINEMASTER SWITCH CORP.
by: Fuss & O'Neill, Inc.
WELL DATA: Units: ft
AQUIFER: UNCONFINED
THICKNESS: 48.92
SCREEN: top: 102.0 base: 112.0
DIAMETER: casing: .1660 intake: .6660
DEPTH: Water Table: 63.78 TD: 112.7

Well Slug Test Data

Well: MW-34T

LINEMASTER SWITCH CORP.

WOODSTOCK, CONNECTICUT

DATA SET: MW34T

CLIENT:	LINEMASTER SWITCH CORP.	DATE:	31-OCT-95
LOCATION:	LINEMASTER SWITCH CORP.	WELL NO.:	MW-34T
COUNTY:	WOODSTOCK, CONNECTICUT	WELL DEPTH:	112.70 ft
PROJECT:	Well Slug Test Data	WATER TABLE:	63.780 ft
AQUIFER:	UNCONFINED	THICKNESS:	48.92 ft
INTAKE RADIUS:	0.333 ft	CASING RADIUS:	0.083 ft
SCREEN TOP:	102.000 ft	SCREEN BASE:	112.00 ft
INITIAL HEAD:	1.300 ft	TRANS. RATIO:	1.0000

MODEL PARAMETERS:

TRANSMISSIVITY: 0.13733square ft/day

CONDUCTIVITY: 0.00281 ft/day

MODEL TYPE: UNCONFINED PARTIALLY PENETRATED AQUIFER (Bouwer & Rice)

No.	TIME (mins)	Head, H (ft) DATA	SYNTHETIC	DIFFERENCE (percent)
1	0.110	1.29		
2	0.120	1.26		
3	0.130	1.24		
4	0.140	1.23		
5	0.150	1.23		
6	0.160	1.21		
7	0.190	1.20		
8	0.200	1.21	1.17	3.61
9	0.230	1.20	1.17	2.33
10	0.273	1.18	1.17	1.01
11	0.330	1.18	1.17	1.01
12	0.450	1.16	1.17	-0.329
13	0.500	1.16	1.17	-0.326
14	0.750	1.16	1.17	-0.310
15	1.60	1.15	1.17	-1.64
16	5.00	1.15	1.16	-1.42
17	7.00	1.15	1.16	-1.29
18	12.00	1.15	1.16	-0.961
19	22.00	1.13	1.15	-1.71
20	46.00	1.12	1.13	-1.57
21	68.00	1.12	1.12	-0.129
22	94.00	1.12	1.10	1.55

CURRENT RESOLUTION MATRIIX NOT AVAILABLE

APPENDIX B
ONE-DIMENSIONAL AIR FLOW MODEL

OVERBURDEN GEOLOGY AND PHYSICAL CHARACTERISTICS
TECHNICAL MEMORANDUM
LINEMASTER SWITCH CORPORATION
WOODSTOCK CONNECTICUT
FEBRUARY 1996

FUSS&O'NEILL consulting engineers	PREPARED BY	DATE	CHECKED BY	DATE	PROJECT N
	CTE	1/27/96	HTK	2/6/96	86-088

Linemaster - One - Dimensional Model

SHEET NO. 1 of 3

<u>Purpose:</u> Develop a solution for the air flow equation given as follows:

$$\dot{m} = \frac{k}{\mu} A \rho \frac{dP}{dz}$$

where:

$\dot{m}$ = Mass flow rate

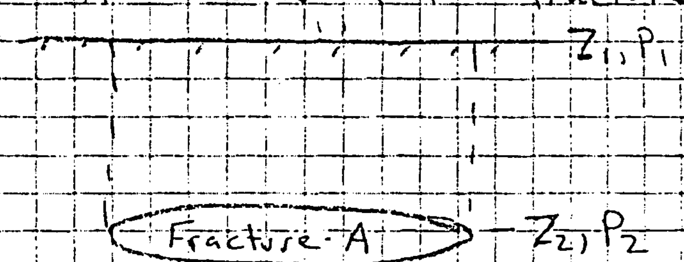
K = air permeability of the soil

ρ = air density

dP = change in pressure

dz = change in elevation

A = area of the fracture



<u>Assumptions:</u>

1. The fracture plane is at constant pressure
2. Ideal gas law applies
3. Steady state conditions $\therefore$ mass rate, $\dot{m}$ is constant

Calculations:

1. Identify Constants

$$\dot{m} = \frac{K}{\mu} A \rho \frac{dP}{dz}$$

In this equation, K, μ, and A are constants, and ρ is a function of pressure.

$$\therefore \frac{K}{\mu} A = C_1$$

2. Since ρ is a function of P, use the ideal gas law to solve for ρ:

$$\rho = \frac{M_w P}{RT}$$

M_w = Molecular weight, T = Temp
R = ideal gas const.

3. Substitute C_1 and ρ

$$\dot{m} = C_1 \frac{M_w P}{RT} \frac{dP}{dz}$$

4. Under standard conditions, T is a constant!

$$C_2 = C_1 \frac{M_w}{RT}$$

$$\therefore \dot{m} = C_2 P \frac{dP}{dz}$$

5. Rearrange the equation and solve

$$\dot{m}\, dz = C_2 P\, dP$$

$$\therefore \dot{m} \int_{z_2}^{z_1} dz = C_2 \int_{P_2}^{P_1} P\, dP$$

$$\therefore \dot{m} z \Big|_{z_1}^{z_2} = \frac{C_2}{2} P^2 \Big|_{P_2}^{P_1}$$

Linemaster - One Dimensional Model

$$\therefore \dot{m}\,(Z_2 - Z_1) = \frac{C_2}{2}\left(P_2^2 - P_1^2\right)$$

6. Solve for $\dot{m}$

Use the ideal gas law under standard conditions to solve for $\dot{m}$

$$PV = nRT \qquad n = \frac{m}{M_w}$$

$$PV = \frac{mRT}{M_w} \;\rightarrow\; \dot{m} = \frac{m}{t}$$

$$\therefore \frac{PV}{t} = \frac{mRT}{t\,M_w}$$

$$\frac{V}{t} = Q \text{ - Air Flow}$$

$$\therefore PQ = \frac{\dot{m}RT}{M_w} \;\rightarrow\; Q = \frac{\dot{m}RT}{P\,M_w}$$

under standard conditions

$$Q = \frac{\dot{m}RT_s}{P_s M_w} \;\rightarrow\; \dot{m} = \frac{Q P_s M_w}{RT_s}$$

7. Substitute for $\dot{m}$ and C_2 and solve for Q

$$Q\,\frac{P_s M_w}{R\,T_s}(Z_2 - Z_1) = \overbrace{\overbrace{\frac{KA}{\mu}}^{C_1}\,\frac{M_w}{R\,T_s}}^{C_2}\,\frac{1}{2}\left(P_2^2 - P_1^2\right)$$

$$Q\,P_s(Z_2 - Z_1) = \frac{KA}{2\mu}\left(P_2^2 - P_1^2\right)$$

$$\therefore \boxed{Q = \frac{KA}{2\mu P_s (Z_2 - Z_1)}\left(P_2^2 - P_1^2\right)}$$

Linemaster - One Dimensional Model

Purpose - check spreadsheet

$$K = \frac{2 \mu P_{atm} Q \Delta z}{A (P_{atm}^2 - P_{vac}^2)}$$

$A = 554\ ft^2 = 514,683\ cm^2$

$\mu = 1.76 \times 10^{-4}\ gm/cm\text{-}s$

$P_{atm} = 1.013 \times 10^6\ gm/cm\text{-}s^2$

$\Delta z = 8\ ft = 244\ cm$

$Q = 24.7\ ft^3/min \times \left(\frac{30.48\ cm}{ft}\right)^3 \times \frac{1\ min}{60\ s} = 11,657\ cm^3/sec$

$P_{vac} = 49.8''\ H_2O \times \frac{1\ atm}{407.8''\ H_2O} = 0.122\ atm\ vac$

$= 0.878\ atm$

$= 889,294\ gm/cm\text{-}s^2$

$$K = \frac{2 (1.76 \times 10^{-4})(1.013 \times 10^6)(11,657)(244)}{514,683 ((1.013 \times 10^6)^2 - (889,294)^2)}$$

$$= \frac{1.014 \times 10^9}{1.21 \times 10^{17}} = 8.37 \times 10^{-9}\ cm^2$$

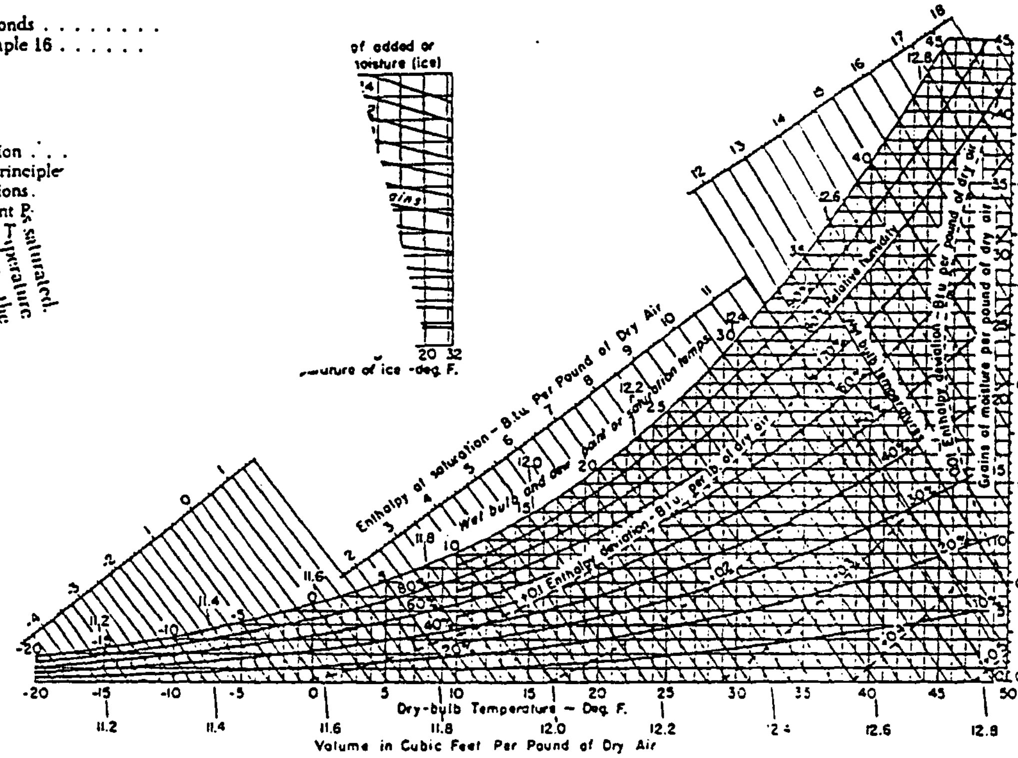


FIG. 12-1 Psychrometric chart—low temperatures. Barometric pressure, 29.92 inHg

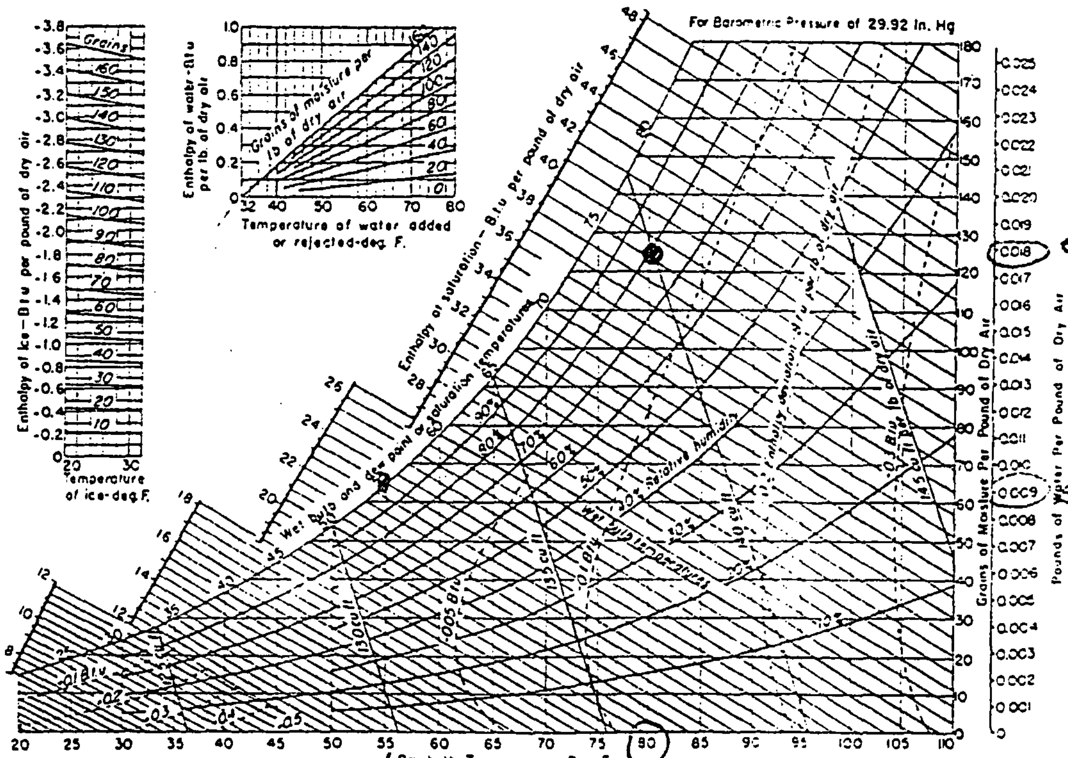


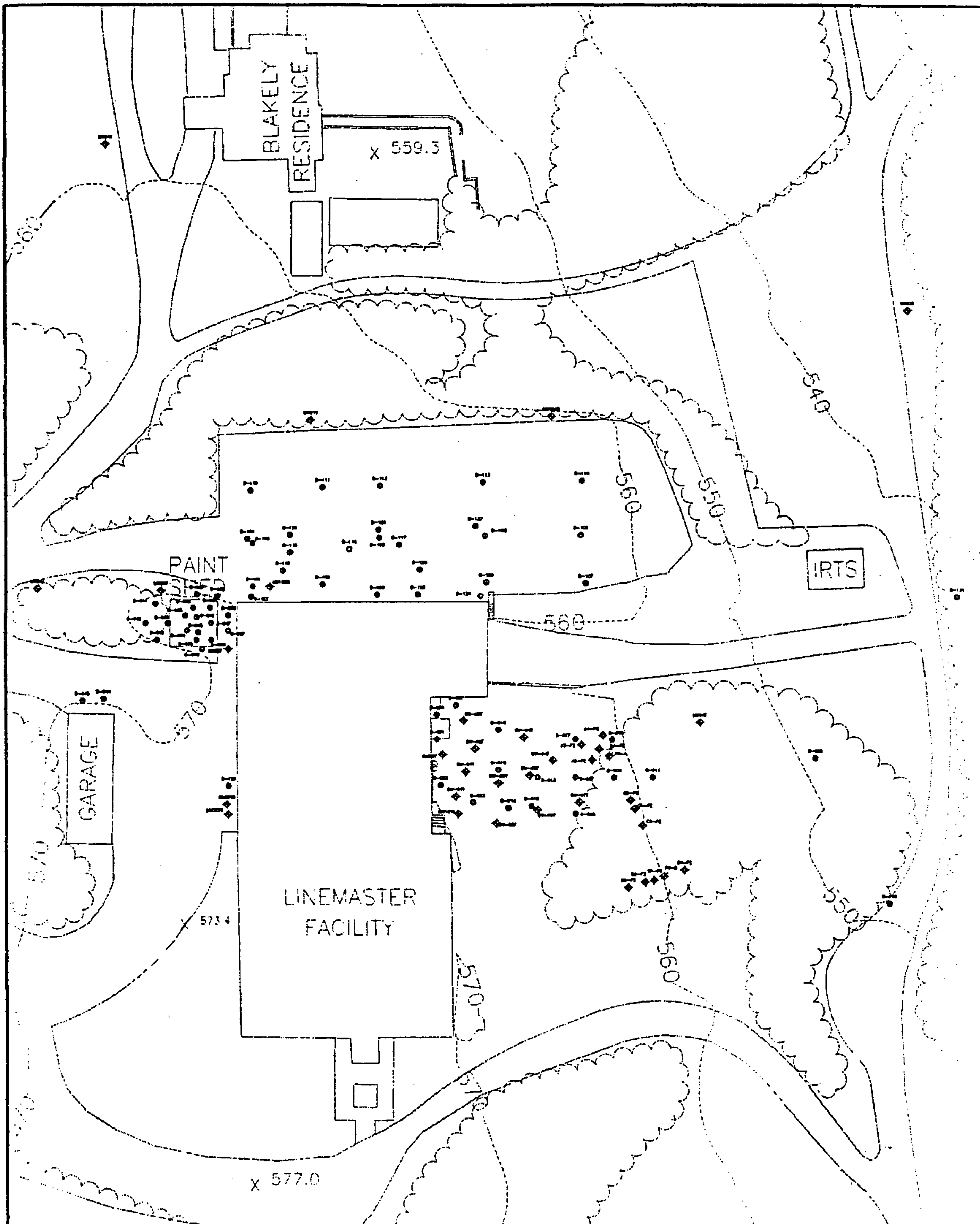
FIG. 12-2 Psychrometric chart—medium temperatures. Barometric pressure, 29.92 inHg

APPENDIX B

APPENDIX B

MARCH 6, 1996 PRESENTATION MATERIAL

- GEOLOGY
- NATURE/EXTENT OF SOIL CONTAMINATION
- SOIL PHYSICAL PROPERTIES



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(203) 646-2469

PROJ. MGR:
DESIGNER:
FN:J:\A5\8X11

LINEMASTER SWITCH COPORATION

PLAINE HILL ROAD
WOODSTOCK, CONNECTICUT

PPP:
DATE: MAR.1996
JOB NO: 86086A5

ZONE 1 GEOLOGY

- Glacial Till Deposits

  dense
  poorly-sorted
  heterogeneous: cobbles, gravel, sand, silt, clay

- No continuous coarse-grained lenses

  shallow fill adjacent to building
  B1-PZ "grain-supported" interval

UPPER TILL UNIT

"Surface" or Upper Till (Melvin et al.)
natural fractures less well developed than in lower till

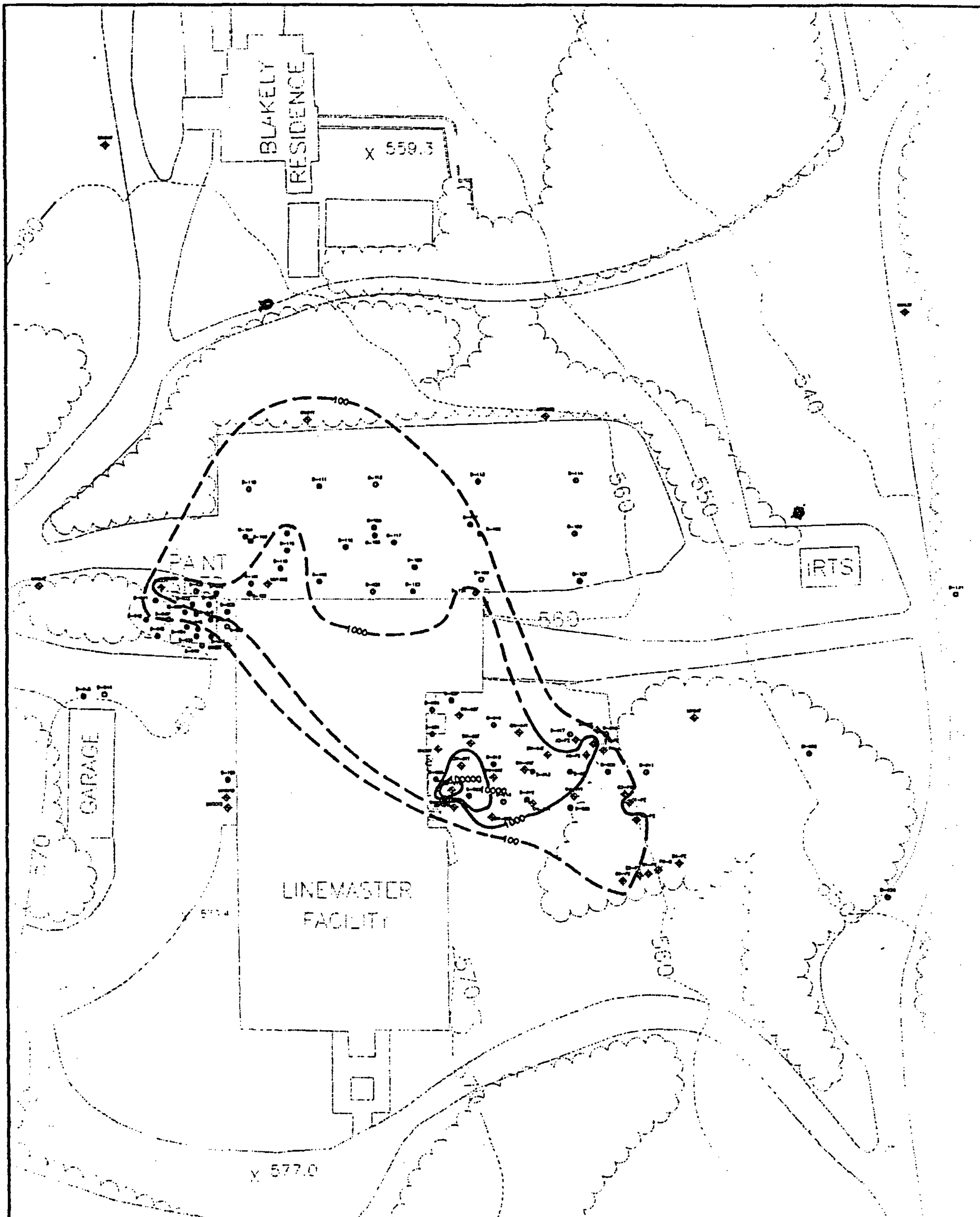
Natural fracture observed at B4-PZ (15.5 ft)

LOWER TILL UNIT

"Drumlin" or Lower Till (Melvin et al.)
natural fractures widely observed, well developed

Melvin et al., 1992a: The Stratigraphy and Hydraulic Properties of Tills in Southern New England; USGS Open File Report 91-481, p. 53.

Melvin et al., 1992b: Hydrogeology of Thick Till Deposits in CT; USGS Open File Report 92-43, p. 43.



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DESIGNER:
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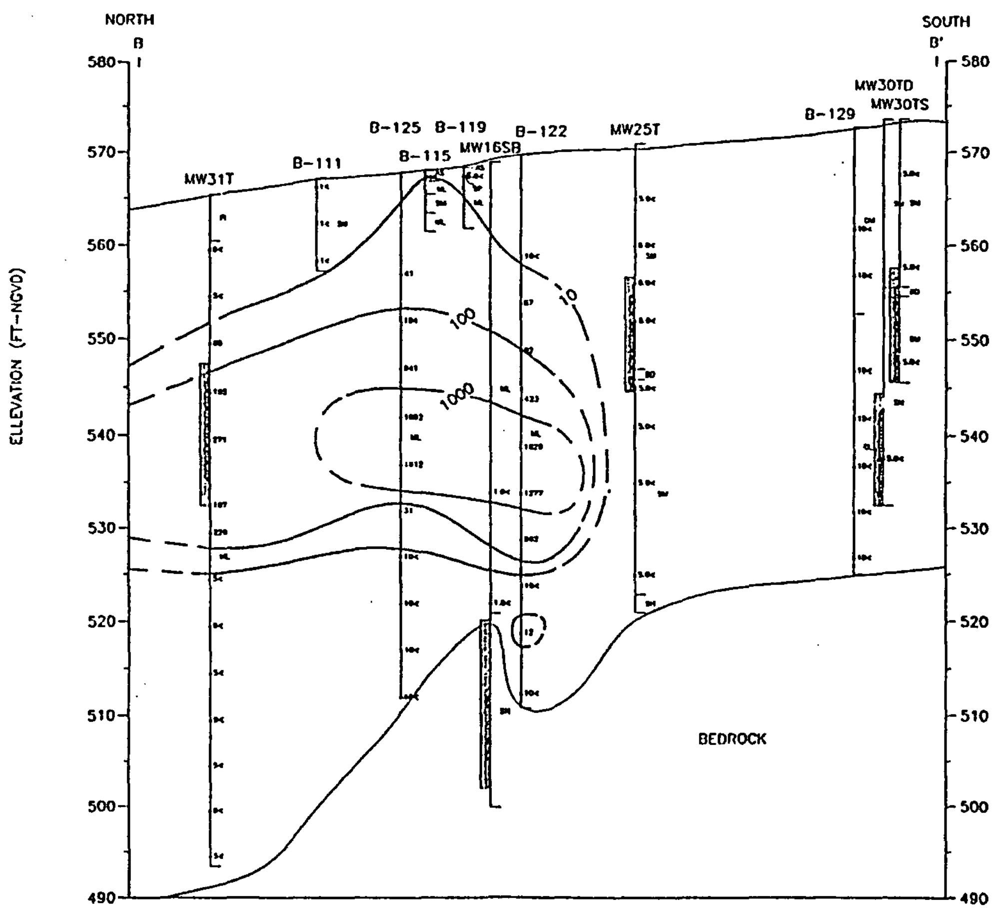
PPP: | DATE: MAR.1996 | JOB NO: 86088A5



LEGEND

MONITORING WELL WITH SCREENED INTERVAL AND FILTER PACK

SM - USCS TYPE SOIL

TCE CONCENTRATION IN SOIL (ug/kg)

SOIL BORING TCE CONCENTRATION IN SOIL (ug/kg)

SM - USCS TYPE SOIL

10 — — — TCE CONCENTRATION ISOPLETH (SOIL)(ug/kg)

HORIZONTAL SCALE: 1" = 40'
VERTICAL SCALE: 1" = 13.5'

NOTE:
TCE CONCENTRATIONS IN SOIL AT WELL MW-16SB WERE NOT USED FOR CONTOURING.
WELL MW-16sb AND BORINGS B-115, B-119 AND B-111 WERE PROJECTED TO THE CROSS SECTION LINE AS SHOWN ON FIGURE 6.1

FIGURE 4.4

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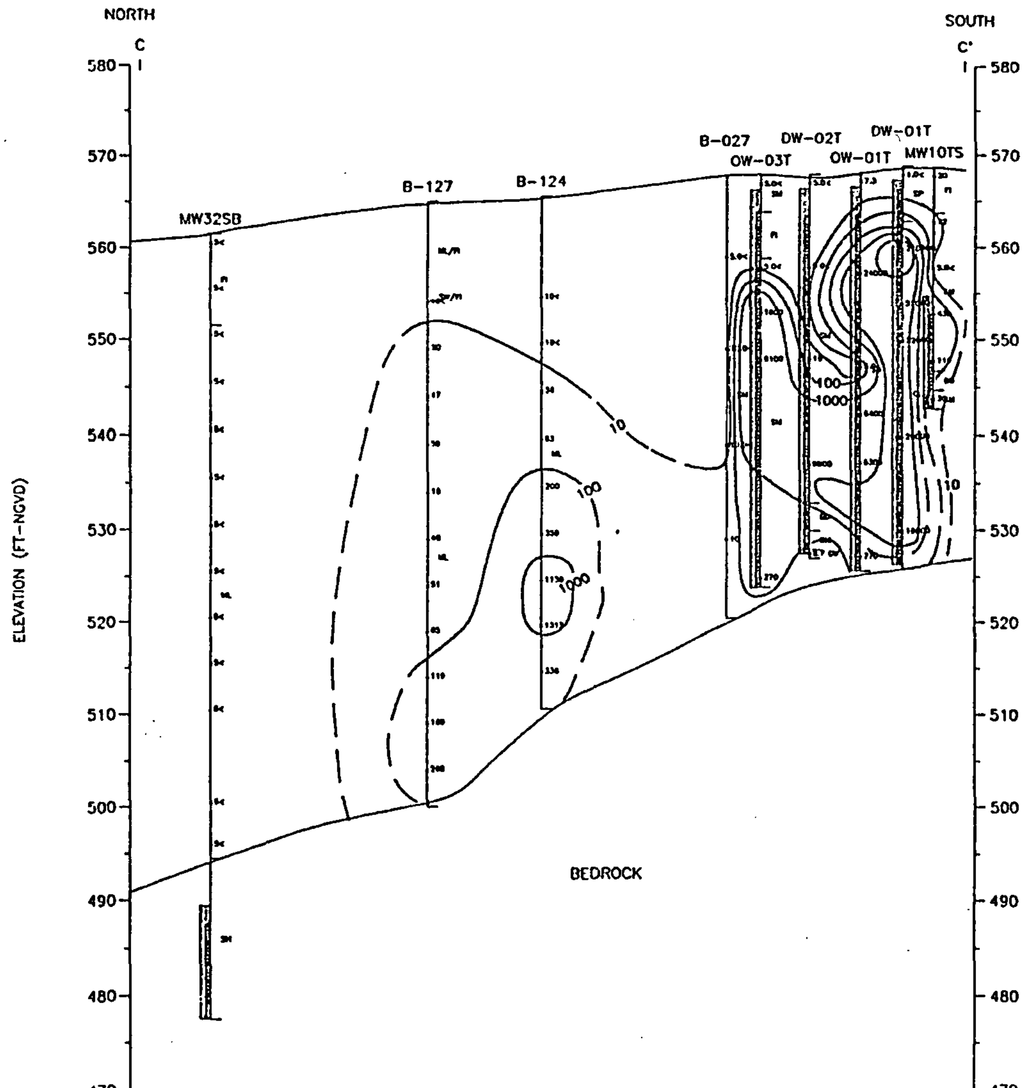
CROSS-SECTION B-B'
ZONE I DELINEATION REPORT



LEGEND

MONITORING WELL WITH SCREENED INTERVAL AND FILTER PACK

SM – USCS SOIL TYPE

TCE CONCENTRATION IN SOIL (ug/kg)

SOIL BORING TCE CONCENTRATION IN SOIL (ug/kg)

SM – USCS SOIL TYPE

10 ——— — — TCE CONCENTRATION ISOPLETH (SOIL)(ug/kg)

HORIZONTAL SCALE: 1" = 40'
VERTICAL SCALE: 1" = 13.5'

FIGURE 4.5

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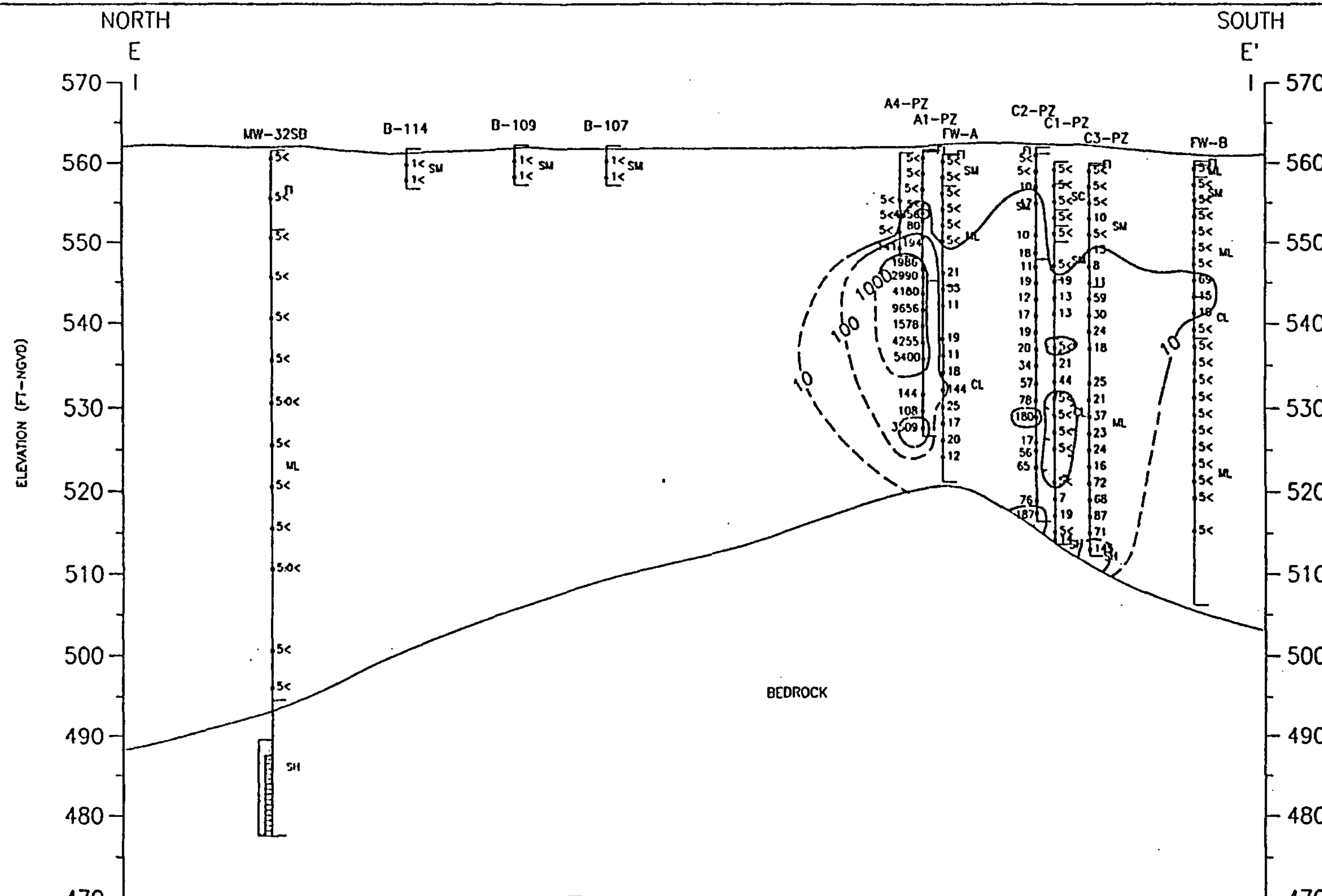
CROSS-SECTION C-C'
ZONE I DELINEATION REPORT
LINEMASTER SWITCH CORPORATION



LEGEND

MONITORING WELL WITH SCREENED INTERVAL AND FILTER PACK

SM – USCS SOIL TYPE

TCE CONCENTRATION IN SOIL (ug/kg)

SOIL BORING TCE CONCENTRATION IN SOIL (ug/kg)

SM – USCS SOIL TYPE

TCE CONCENTRATION ISOPLETH (SOIL)(ug/kg)

HORIZONTAL SCALE: 1" = 30'
VERTICAL SCALE: 1" = 13.5'

NOTE:
SOIL BORINGS B-109 AND B-107 WERE PROJECTED TO THE CROSS SECTION [illegible]

FIGURE 4.7

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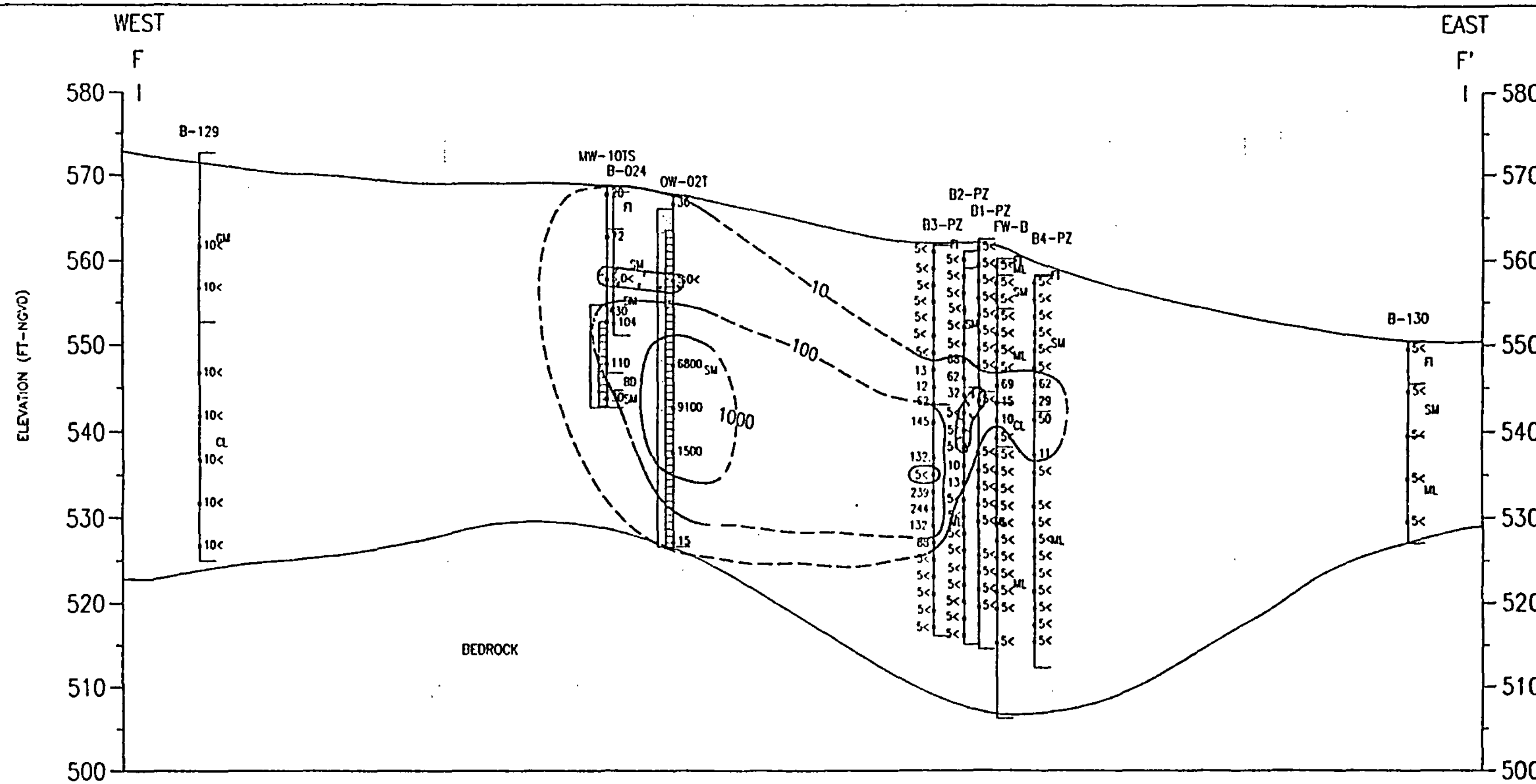
646-[illegible]

CROSS-SECTION E-E'
ZONE I DELINEATION REPORT
[illegible]TION



LEGEND

MONITORING WELL WITH SCREENED INTERVAL AND FILTER PACK

SM – USCS SOIL TYPE

TCE CONCENTRATION IN SOIL (ug/kg)

SOIL BORING TCE CONCENTRATION IN SOIL (ug/kg)

SM – USCS SOIL TYPE

10 ——— — — TCE CONCENTRATION ISOPLETH (SOIL)(ug/kg)

HORIZONTAL SCALE: 1" = 30'
VERTICAL SCALE: 1" = 13.5'

FIGURE 4.8

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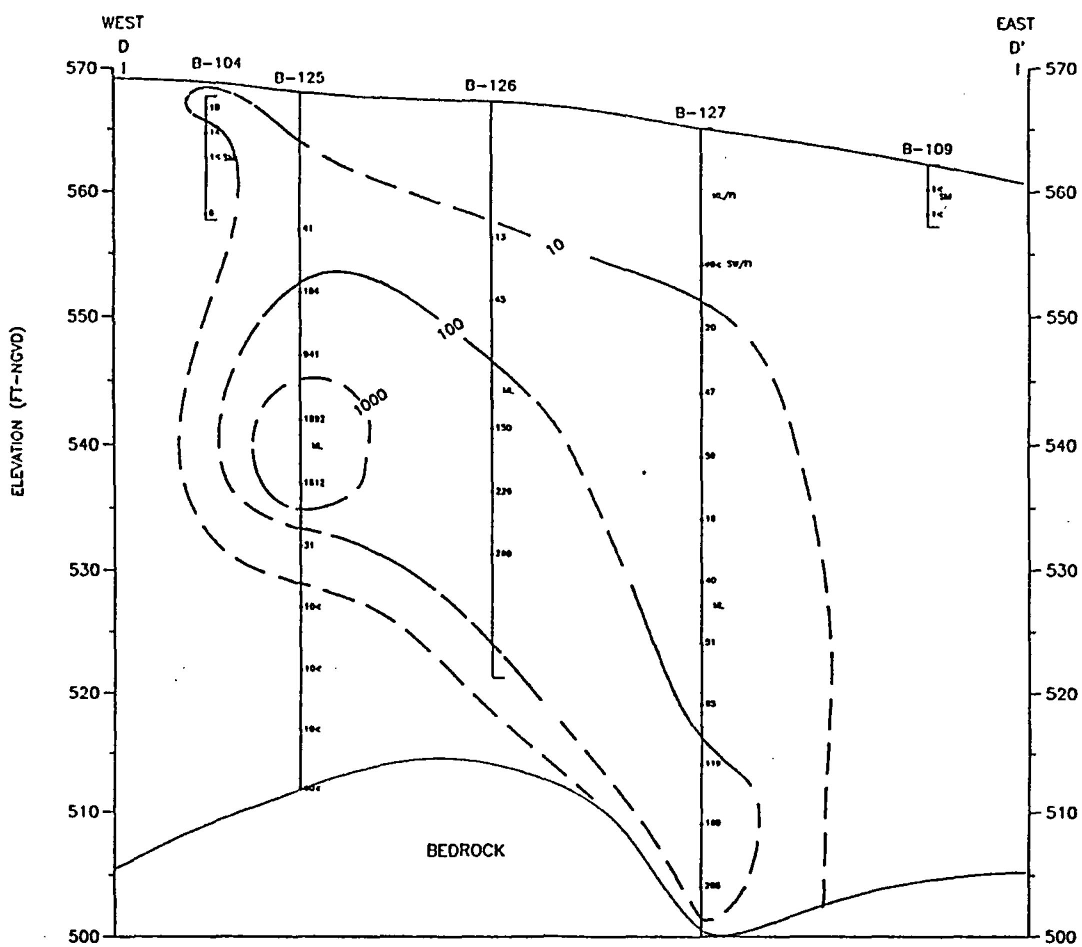
CROSS-SECTION F-F'
ZONE I DELINEATION REPORT
LINEMASTER SWITCH CORPORATION

PLAINE HILL ROAD WOODSTOCK, CONNECTICUT
DATE: FEB 1996



LEGEND

SOIL BORING
TCE CONCENTRATION
IN SOIL (ug/kg)

SM - USCS SOIL TYPE

10 — — — TCE CONCENTRATION ISOPLETH (SOIL)(ug/kg)

HORIZONTAL SCALE: 1" = 30'
VERTICAL SCALE: 1" = 10'

NOTE:
SOIL BORINGS B-126 AND B-127 WERE PROJECTED ONTO THE CROSS SECTION LINE AS SHOWN ON LINE OF SECTION ON FIGURE 6.1

FIGURE 4.6

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CROSS-SECTION D-D'
ZONE I DELINEATION REPORT
LINEMASTER SWITCH CORPORATION

GRAIN SIZE DISTRIBUTION
UPPER and LOWER TILL

PARTICLE CLASS	UPPER TILL AVERAGE (%)	LOWER TILL AVERAGE (%)
GRAVEL	11	22
SAND	39	35
SILT	30	25
CLAY	20	18

NOTES:
Grain size distribution percentages are by weight.
Data summarized from Table 3.1 of 2/28/96 Tech. Memo.

OVERBURDEN PHYSICAL CHARACTERISTICS

	Dry Bulk Density (g/cm3)	Initial Moisture Content		Specific Gravity	Porosity (%)
		Gravimetric (%)	Volumetric (%)		
UPPER TILL AVERAGE	2.00	11.8	21.3	2.66	25.2
LOWER TILL AVERAGE	2.05	11.7	21.9	2.69	23.9

NOTES: Data summarized from Table 3.2 of Technical Memorandum

METHODS OF PERMEABILITY DETERMINATION

HYDRAULIC CONDUCTIVITY

Horizontal

Slug Tests

DW-1t Pumping Test

Hydraulic Fracturing Pilot Test
drawdown and recovery data analyses

Vertical

Laboratory permeameter testing

AIR PERMEABILITY

Horizontal

Control Well testing

Vertical

Control Well testing

Fracture Well SVE testing phases

Laboratory VEQ testing

SCALE DEPENDENCY OF PERMEABILITY MEASUREMENT

Rovey and Cherkauer, 1995

- Hydraulic conductivity increases with scale of measurement
- High rate of increase in K indicates substantial secondary porosity

Bruner and Luttenegger, 1994

- Hydraulic conductivity increases with scale of measurement

  Lab $<<$ Bailer (slug) Tests $<$ Pumping Tests

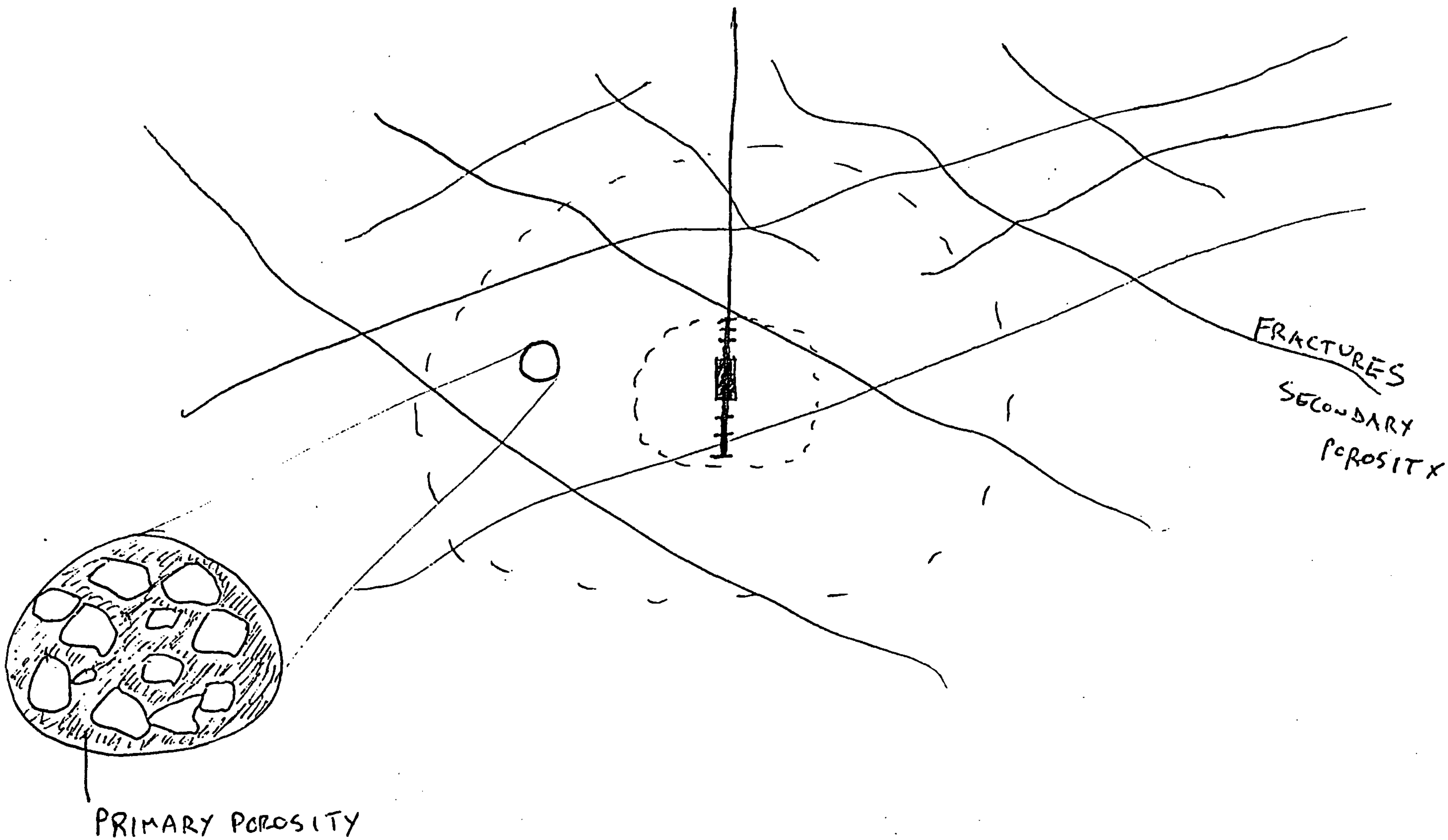
- Starting estimate of field K should be 3 OOM $>$ than lab K

OVERALL CONCLUSIONS

- Laboratory methods estimate the matrix (or primary) permeability
- Field methods estimate the bulk (or secondary) permeability

Bruner and Luttenegger, 1994: Measurement of Saturated Hydraulic Conductivity in Fine-grained Glacial Tills in Iowa; In Daniel and Trautwein (eds), Hydraulic Conductivity and Waster Contaminant Transport in Soils. ASTM STP 1142.

Rovey and Cherkauer, 1995: Scale Dependency of Hydraulic Conductivity Measurements; Groundwater, v. 33, n. 5, pp 769-780.


FRACTURES
SECONDARY
POROSITY
PRIMARY POROSITY

HYDRAULIC CONDUCTIVITY SUMMARY TABLE

Depth of Soil Zone	Horizontal Hydraulic Conductivity (ft/day)		Vertical Hydraulic Conductivity (ft/day)	
	Laboratory	Field	Laboratory	Field
0 to 5 feet (Upper Till)	N/A	N/A	**3.0×10^{-2}**	N/A
5 to 8 feet (Upper Till)	N/A	N/A	N/A	N/A
8 to 18 feet (Upper Till)	N/A	**1.5×10^{-2}**	**8.4×10^{-5}**	N/A
Greater than 20 feet (Lower Till)	N/A	**3.0×10^{-3}**	**5.0×10^{-5}**	N/A

Source: Table 4.3 in 2/28/96 Technical Memorandum

AIR PERMEABILITY SUMMARY TABLE

Depth of Soil Zone	Horizontal Air Permeability (cm^2)		Vertical Air Permeability (cm^2)	
	Laboratory	Field	Laboratory	Field
0 to 5 feet (Upper Till)	N/A	1.5×10^{-9}	N/A	4.2×10^{-8}
5 to 8 feet (Upper Till)	N/A	N/A	N/A	2.2×10^{-9}
8 to 18 feet (Upper Till)	N/A	N/A	4.9×10^{-11}	1.5×10^{-9}
Greater than 20 feet (Lower Till)	N/A	N/A	6.3×10^{-12}	1.6×10^{-10}

Source: Table 5.5 in 2/28/96 Technical Memorandum

PERMEABILITY CONCLUSIONS

- K and k decrease with increasing depth, regardless of orientation or measurment method;

- Upper Till is up to 1 order of magnitude more permeable than Lower Till, based on:

  K_v lab testing

  K_h field testing

  k_v field and lab testing

- Upper Till (0-5 ft) is anisotropic based on control well air permeability testing:

  $k_v > k_h$

- Scale Dependency Effects are present:

  Field $k_v >> $ lab k_v,

and suggest that secondary porosity (i.e. fractures) is significant

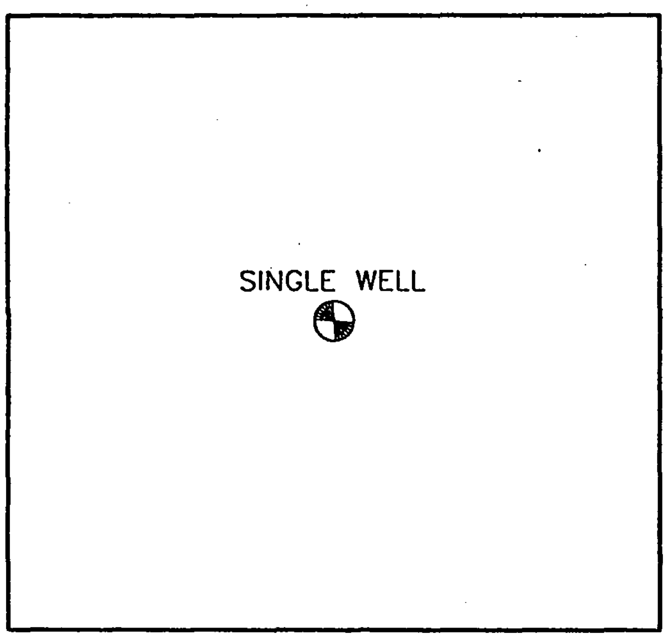
FEASIBILITY

- DEWATERING
- CONTAMINANT TRANSPORT MODELS
  - ADVECTION
  - DIFFUSION
- SUMMARY

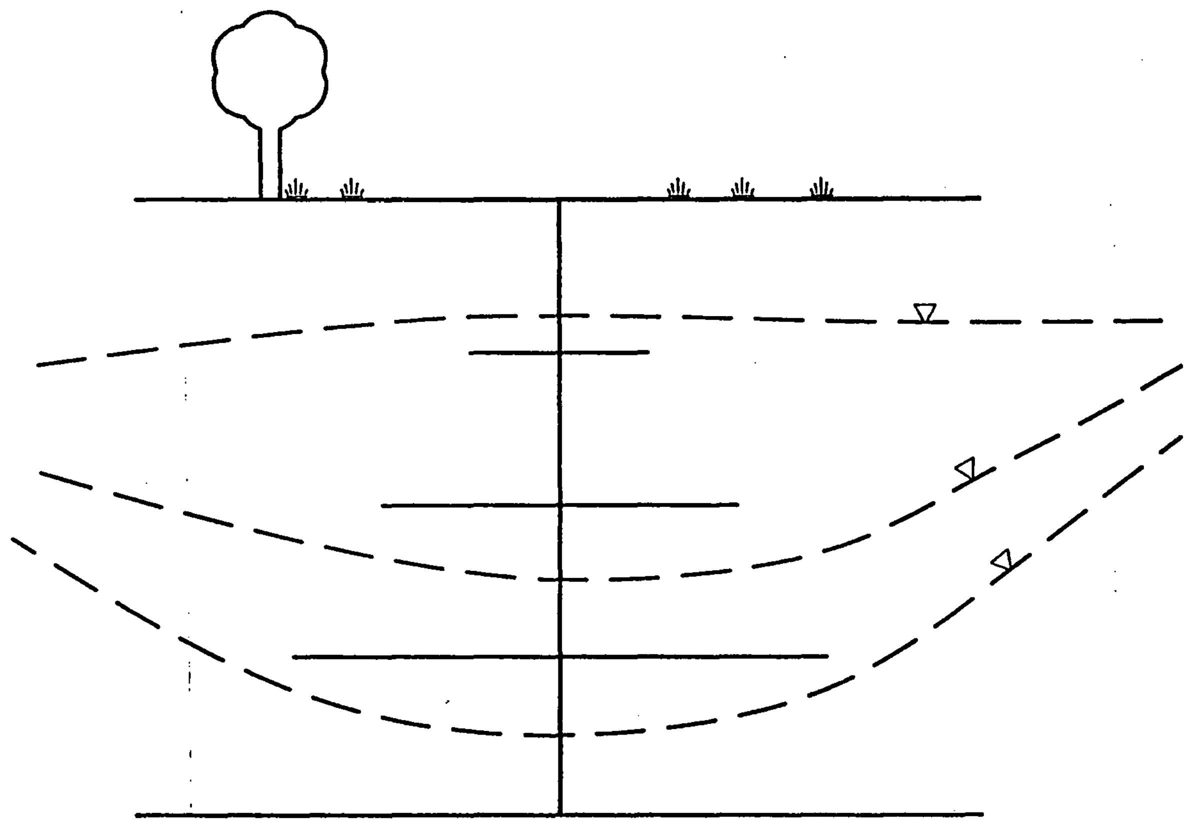
LINEMASTER SWITCH CORPORATION
ZONE 1 REMEDIATION FEASIBILITY ISSUES

DEWATERING

- MODFLOW
- SIMULATED FLOW AND DEWATERING TO A SINGLE FRACTURED WELL (NOVEMBER 1995 REPORT)



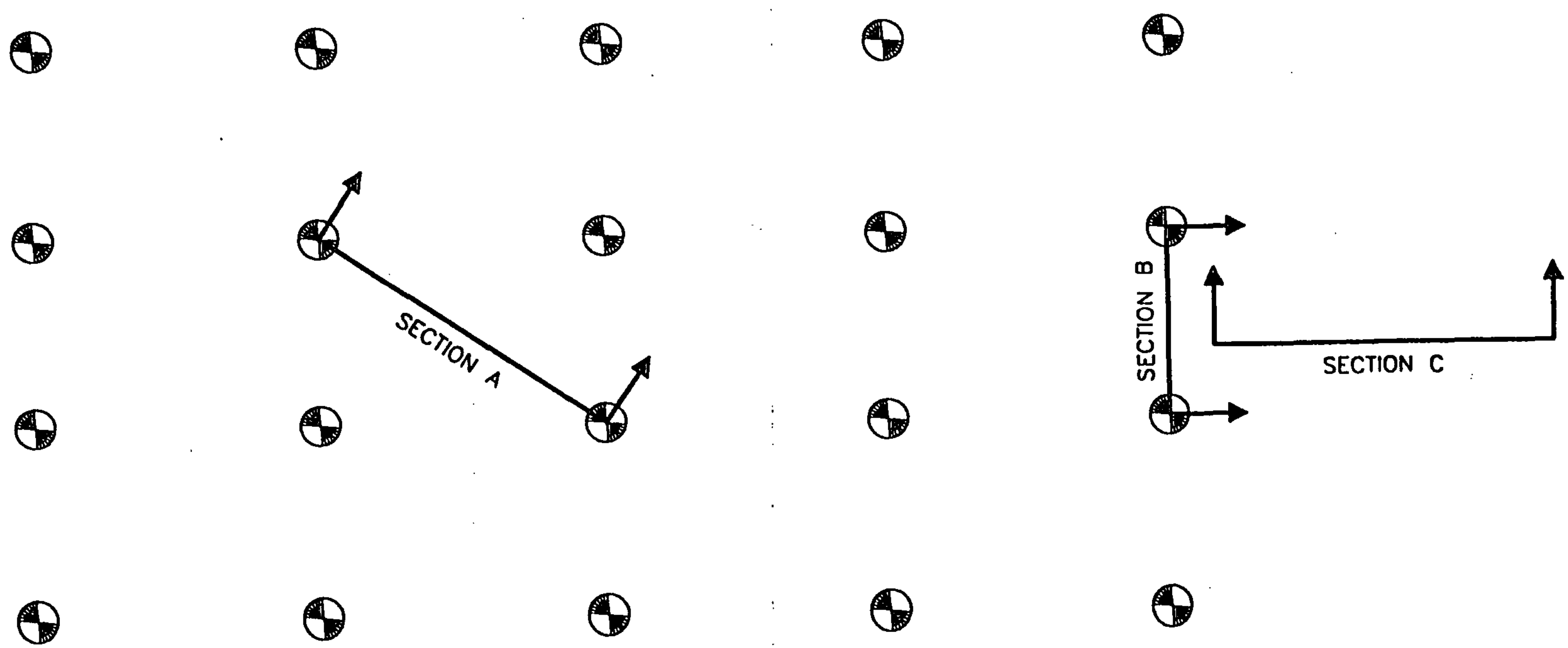
PLAN VIEW



SECTION VIEW

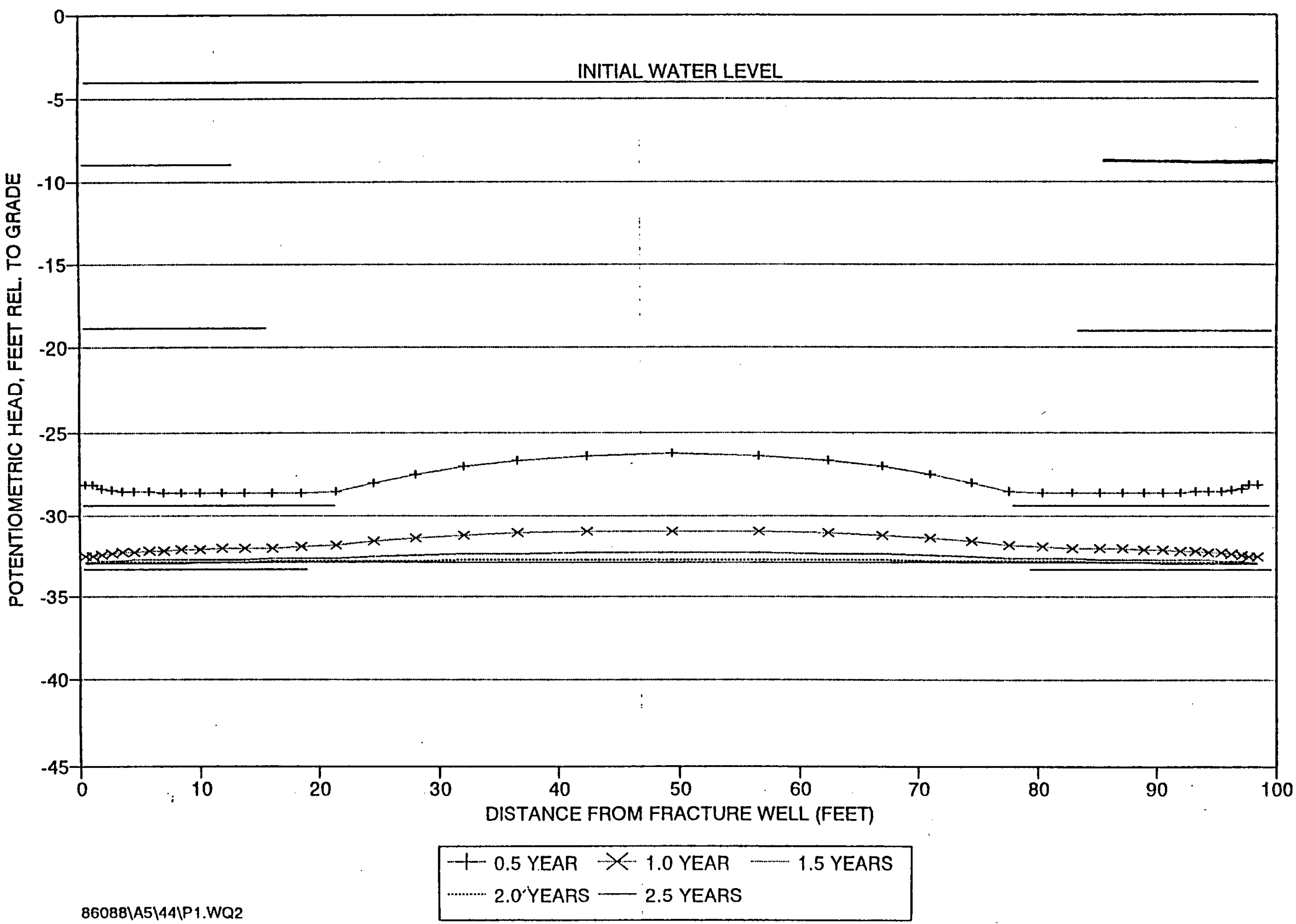
DEWATERING

- CALIBRATED MODEL USING PILOT TEST DATA
- APPLY MODEL TO NETWORK OF FRACTURED WELLS
- DETAILS WILL BE AVAILABLE WITH CONCEPTUAL DESIGN REPORT



PLAN VIEW (MODELLED 70 FOOT WELL SPACING)

SECTION A
FOUR FRACTURE WELL - FW-A, 70 ON-CENTER SPACING



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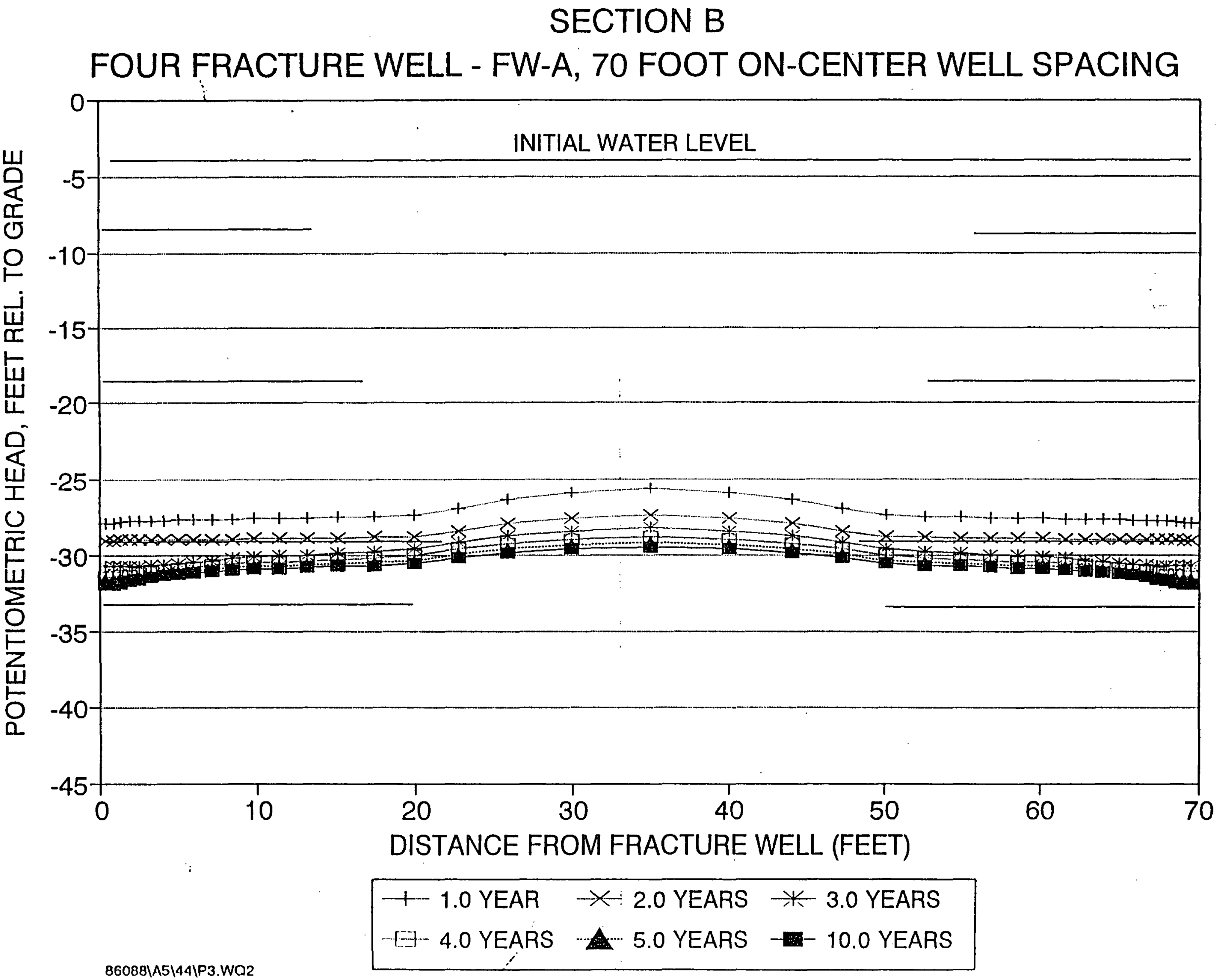
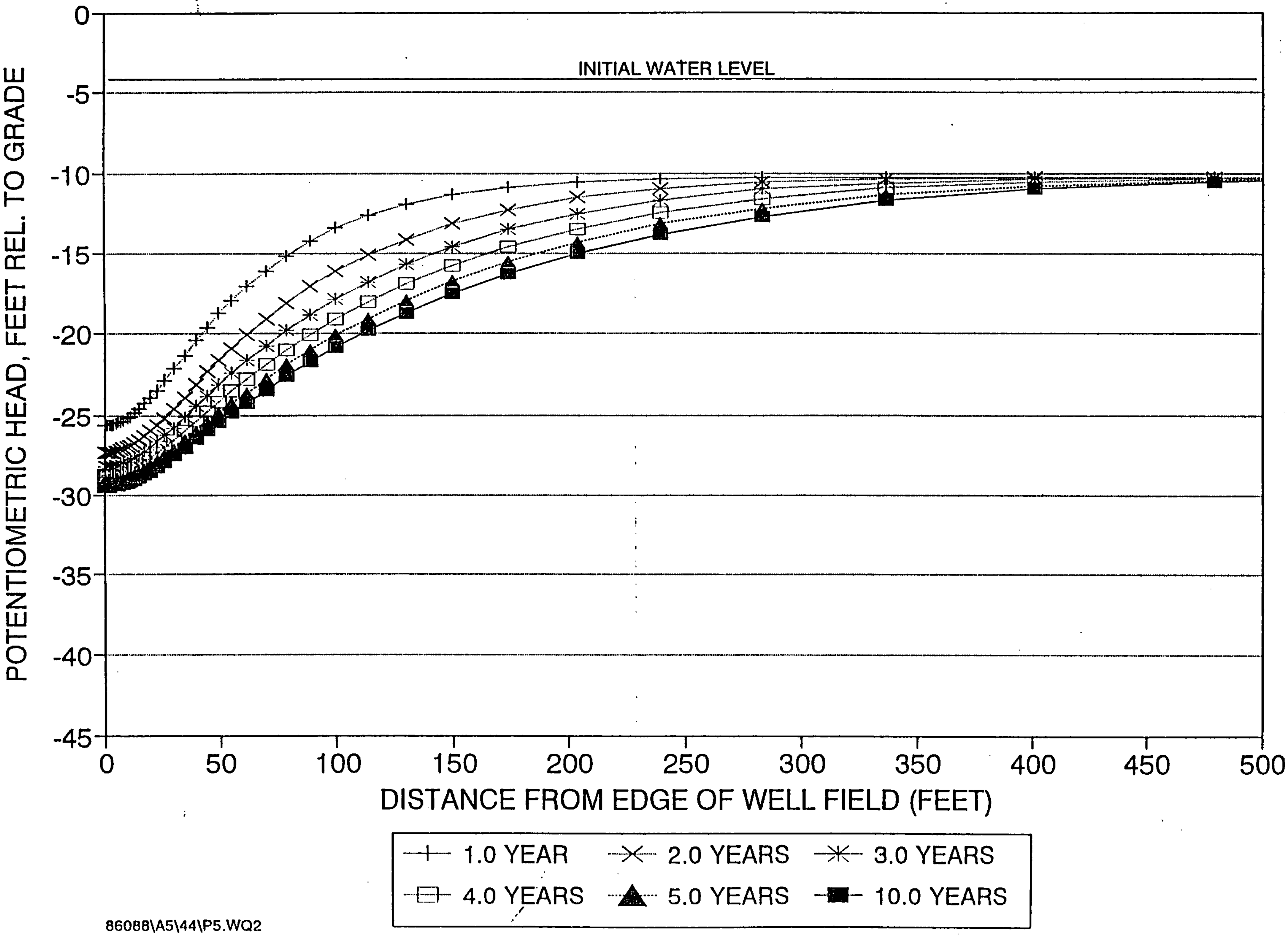


SECTION B
FOUR FRACTURE WELL - FW-A, 70 FOOT ON-CENTER WELL SPACING
INITIAL WATER LEVEL
POTENTIOMETRIC HEAD, FEET REL. TO GRADE
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
0
10
20
30
40
50
60
70
DISTANCE FROM FRACTURE WELL (FEET)
1.0 YEAR
2.0 YEARS
3.0 YEARS
4.0 YEARS
5.0 YEARS
10.0 YEARS
86088\A5\44\P3.WQ2

SECTION C

FOUR FRACTURE WELL - FW-A, 70 FOOT ON-CENTER WELL SPACING



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DEWATERING

- IN THE MIDDLE OF THE WELL FIELD (SECTION A):
  - CAN DEWATER TO THE DEPTH OF THE DEEPEST FRACTURE
  - MOUNDING IS INSIGNIFICANT AFTER A YEAR OR MORE
- AT EDGE OF WELL FIELD (SECTIONS B & C):
  - DEWATERING PROFILE CAN BE PREDICTED
  - CRITERIA ARE AVAILABLE FOR LOCATING WELLS AT EDGE OF ZONE ONE
    - DEPTH OF CONTAMINATION
    - DEPTH OF DEWATERING
- CURRENT MODEL IS CONSERVATIVE
  - THEORETICAL WELL SPACING USED WAS 70 FEET.
    - CAN USE CLOSER (HEXAGONAL PACK) WELL SPACING
  - FRACTURE SPACING IN WELL WAS 10 FEET
    - CAN USE CLOSER FRACTURE SPACING
  - MODEL DOES NOT ACCOUNT FOR VACUUM ENHANCED WATER RECOVERY

CONTAMINANT TRANSPORT - OVERVIEW

- EXAMPLE PERFUME

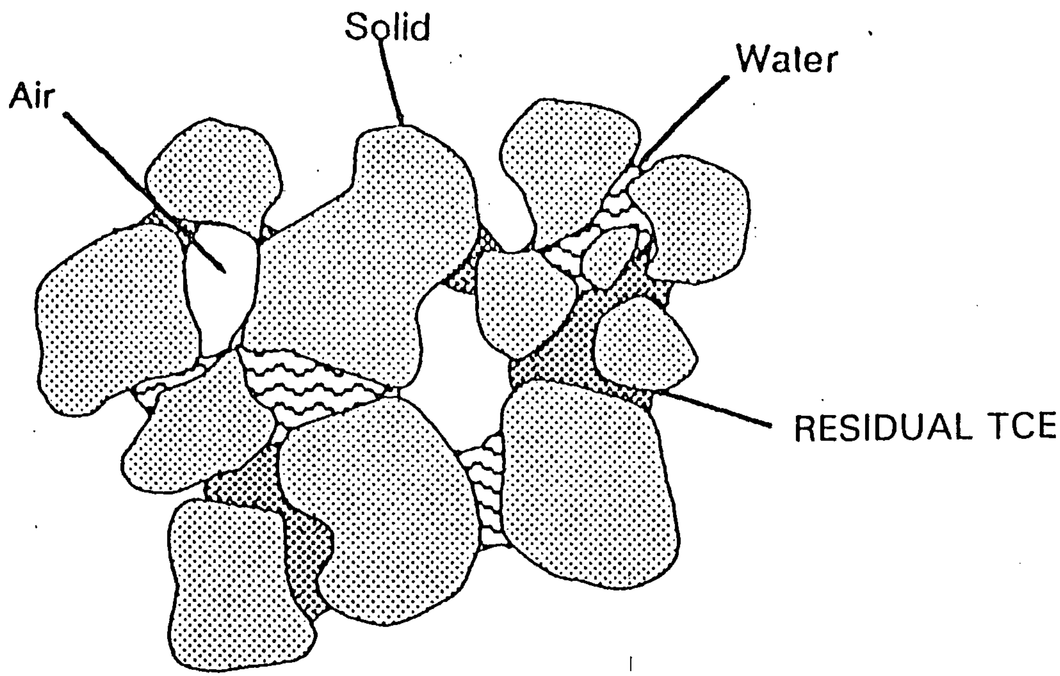
- ADVECTION: TRANSPORT OF CONTAMINANTS THROUGH PHYSICAL MOVEMENT OF FLUID IN WHICH THE CONTAMINANT IS CONTAINED
  - AQUEOUS PHASE (SLOW MOVING)
  - VAPOR PHASE (FASTER MOVING)

- DIFFUSION: TRANSPORT OF CONTAMINANTS WHERE THERE IS NO ADVECTION. DUE TO CONCENTRATION GRADIENT.
  - AQUEOUS PHASE (SLOW DIFFUSION)
  - VAPOR PHASE (FASTER DIFFUSION)

LINEMASTER SWITCH CORPORATION
ZONE 1 REMEDIATION FEASIBILITY ISSUES

CONTAMINANT TRANSPORT - ADVECTION

- SOIL MATRIX:



- CONTAMINANT PARTITIONING - IN TILL MATRIX

  - AQUEOUS PHASE{ C_{WATER} }
  - VAPOR PHASE { $C_{AIR} = H\ C_{WATER}$ }
  - SORBED PHASE { $C_{SORBED,SOIL} = k_d\ C_{WATER}$ }
  - RESIDUAL TCE { $C_{WATER} = C_{SAT,WATER}$, $C_{AIR} = C_{SAT,AIR}$ }

CONTAMINANT TRANSPORT - ADVECTION

- RESIDUAL PRESENT: $C_{AIR} = C_{SAT,AIR}$
- NO RESIDUAL PRESENT, RELATIONSHIP BETWEEN C_{AIR} AND C_{TOTAL}:

$$\frac{C_{AIR}}{C_{TOTAL}} = \frac{1}{\left\{\frac{n-n_a}{H} + \frac{k_d\ \rho_b}{H} + n_a\right\}}$$

WHERE

n IS THE TOTAL POROSITY OF THE SOIL

n_a IS THE AIR FILLED POROSITY OF THE SOIL,

k_d IS THE PARTITIONING COEFFICIENT BETWEEN SOIL AND WATER,

ρ_b IS THE BULK DENSITY OF THE SOIL, AND

H IS THE HENRY'S CONSTANT.

THE ABOVE EXPRESSION IS EQUIVALENT TO EQ. (1) OF DIGIULIO, D.C. 1992 (EPA 540/S-92/004)

- MASS REMOVAL - PRODUCT OF AIR VOLUME AND AIR CONCENTRATION

CONTAMINANT TRANSPORT - ADVECTION

- ADVECTION MODEL INPUTS
  - PHYSICAL PARAMETERS FOR TCE
  - SOIL PROPERTIES DEVELOPED IN TECHNICAL MEMORANDUM
    - TOTAL POROSITY $n_a = 24\%$
    - AIR FILLED POROSITY $n = 3\%$
    - PARTITIONING COEFFICIENT $k_d = 0.26$ cc/gram
    - BULK DENSITY $\rho_b = 2$ gram/cc
- INITIAL AND FINAL CONDITIONS
  - INITIAL CONDITIONS - MORE DIFFICULT AREA TO REMEDIATE
    - 100 TIMES AVERAGE CONCENTRATION OF TCE DESCRIBED IN RI/FS
    - 1.35 LBS TCE/CUBIC FOOT SOIL
    - 1.3 LBS TCE RESIDUAL
    - 0.05 LBS TCE PARTITIONED INTO SOIL, WATER, VAPOR
    - RESIDUAL TCE FILLS LESS THAN 6 PERCENT OF TOTAL VOID SPACE
  - FINAL CONDITIONS: CLEANUP CRITERIA OF 100 UG TCE PER KG SOIL

CONTAMINANT TRANSPORT - ADVECTION

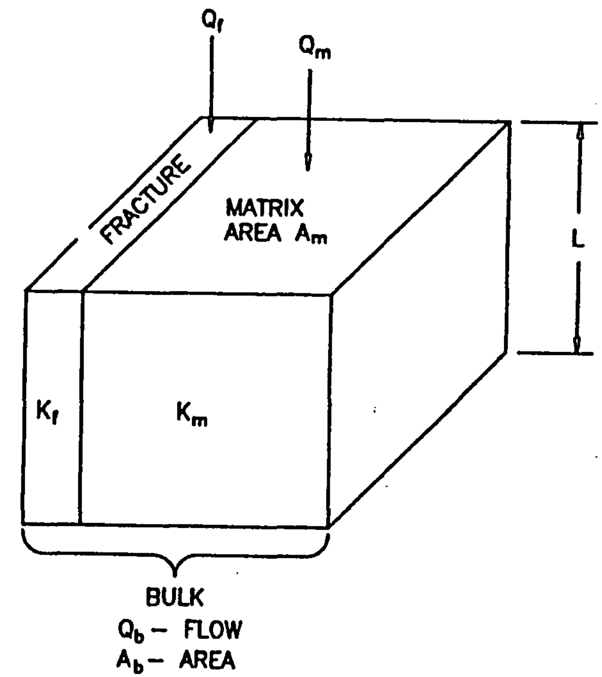
- MODEL SETUP
  - DIMENSION-LESS
  - EXPRESSES CLEANUP TIME IN PORE VOLUME CHANGES
  - PORE VOLUME: VOLUME OF AIR FILLED POROSITY IN SOIL

- MODEL RESULTS
  - DEPLETE RESIDUAL TCE: 2,900 PORE VOLUME CHANGES
  - ACHIEVE REMEDIAL CLEANUP CRITERIA: 900 PORE VOLUME CHANGES
  - TOTAL CLEANUP: 3,800 PORE VOLUME CHANGES
  - PORE VOLUME CHANGE RATE (PVCR) IS FOR TILL MATRIX

- MODEL IS OPTIMISTIC - CLEANUP WILL TAKE LONGER
  - FLOW THROUGH TILL MATRIX IS NOT UNIFORM
  - DIFFUSION LIMITATIONS ARE NOT CONSIDERED

CONTAMINANT TRANSPORT - ADVECTION

- MATRIX VS. BULK TILL PORE VOLUME CHANGE RATES
- CONTROL VOLUME MODEL:



$$\frac{PVCR_m}{PVCR_b} = \frac{\dfrac{Q_m}{V_{air_m}}}{\dfrac{Q_b}{V_{air_b}}} = \frac{\dfrac{A_m k_m \frac{\Delta P}{L}}{A_m L n_{a_m}}}{\dfrac{A_b k_b \frac{\Delta P}{L}}{A_b L n_{a_b}}} = \frac{k_m\ n_{a_b}}{k_b\ n_{a_m}} \approx \frac{k_m}{k_b} = \frac{1}{30}$$

- TOTAL CLEANUP TIME:

3,800 PORE VOLUME CHANGES IN TILL MATRIX

120,000 PORE VOLUME CHANGES IN BULK TILL

CONTAMINANT TRANSPORT - ADVECTION

- DEVELOP CRITERIA FOR PVCR

  - SET CLEAN UP GOAL OF 10 YEARS

  - ASSUME

    - ONE (1) YEAR TO DEWATER

    - TWO (2) MODES OF OPERATION

- REQUIRED PVCR

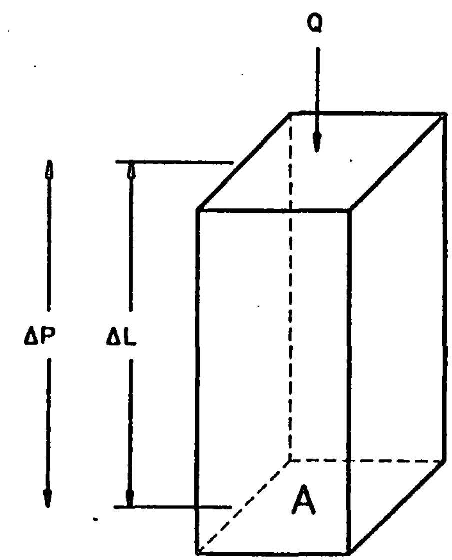
$$PVCR = \frac{120{,}000\ \textit{pore volumes}}{\frac{(10-1)}{2}\ \textit{years}} = 73\ \frac{\textit{pore volumes}}{\textit{day}}$$

CONTAMINANT TRANSPORT - ADVECTION

- ESTIMATE OF PORE VOLUME CHANGE RATE BY:

$$PVCR = \frac{MassFlux_{air}}{Mass_{air}} = \frac{\rho_{air} A \frac{k \Delta P}{\mu \Delta L}}{\rho_{air} A \Delta L n} = \frac{k \Delta P}{\mu n (\Delta L)^2}$$

- WHERE

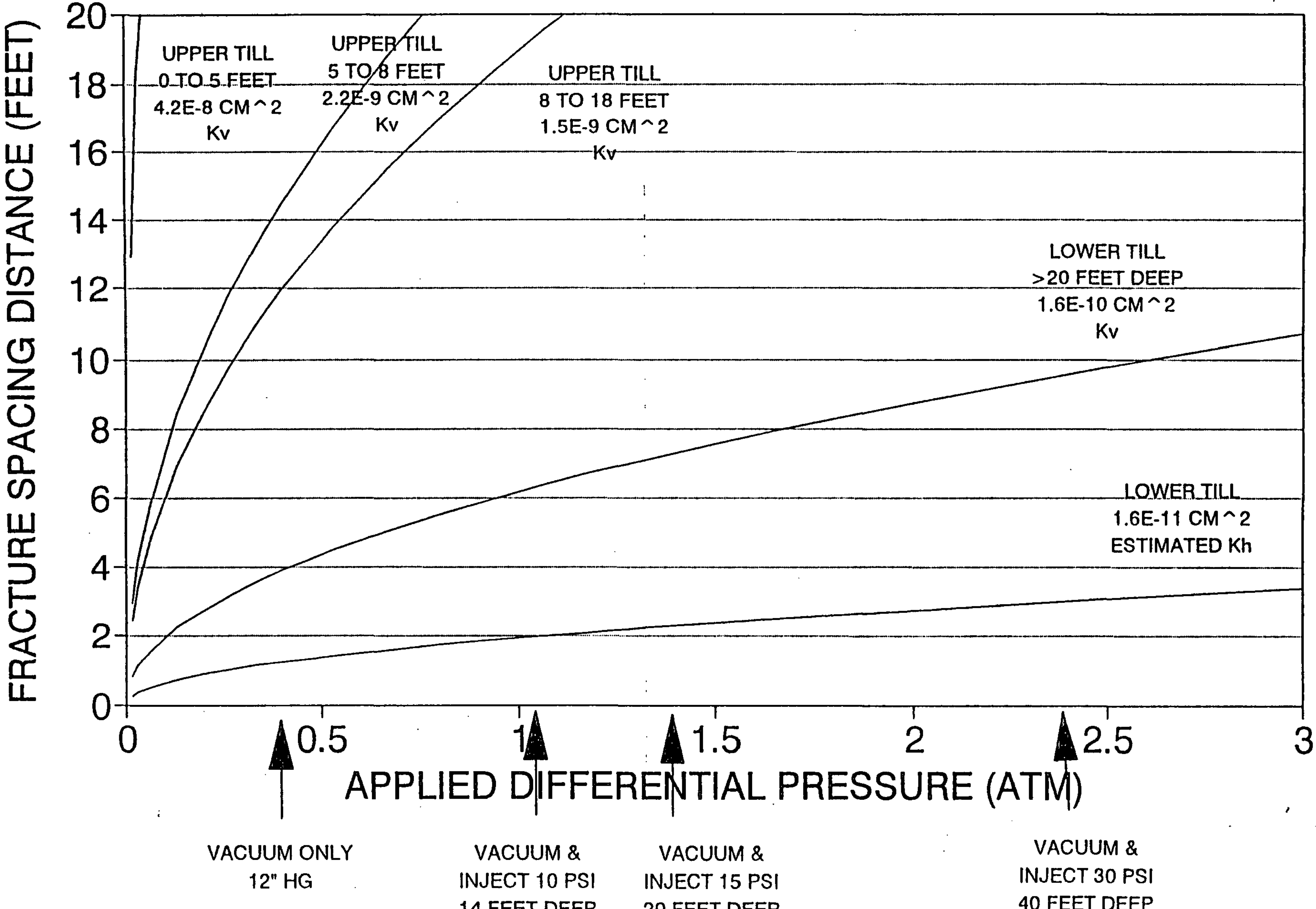


- IF PVCR IS KNOWN, CAN DEVELOP DESIGN CRITERIA:

$$\Delta L = \sqrt{\frac{k_a \, \Delta P}{\mu \; n \; PVCR}}$$

OPTIMAL SCENARIO FOR FRACTURE SPACING

10 Year Minimum Remedial Timeframe, 1 Year to Dewater
3% Air Porosity and 73 PVCs per day



86088\A5\44\TRVLDIST.WQ2

CONTAMINANT TRANSPORT - ADVECTION SUMMARY

- ADVECTION MODEL IS VERY OPTIMISTIC
  - FLOW THROUGH TILL MATRIX IS NOT UNIFORM
  - DIFFUSION LIMITATIONS ARE NOT CONSIDERED
- IF DESIGN CRITERIA PRESENTED ARE MET
  - REMEDIATION WILL TAKE AT LEAST 10 YEARS

CONTAMINANT TRANSPORT - DIFFUSION

- MODEL DIFFUSION OF TCE IN SOIL

- IGNORE ADVECTION

- MASS TRANSPORT IS PROPORTIONAL TO CONCENTRATION GRADIENT

  - VAPOR $\Delta C_{AIR}/\Delta L$

  - WATER $\Delta C_{WATER}/\Delta L$

- PROPORTIONALITY CONSTANT IS THE DIFFUSIVITY

  - VAPOR ONLY $D_{AIR} = 8.2 \times 10^{-2}$ CM²/SEC

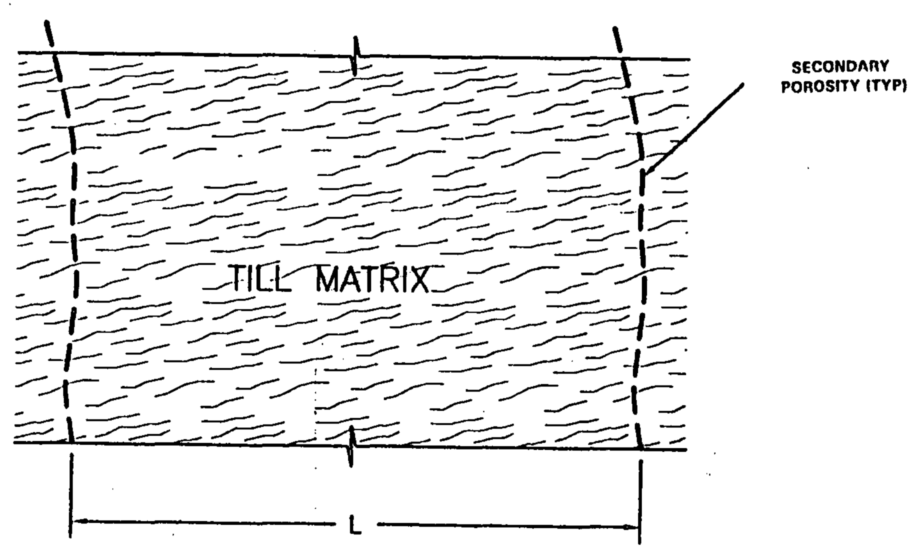
  - WATER ONLY $D_{WATER} = 9.0 \times 10^{-6}$ CM²/SEC

  - COMPOSITE POROUS MEDIA* $D_{COMP} = 3.6 \times 10^{-6}$ CM²/SEC

  * FOR UNSATURATED POROUS MEDIA (McCARTHY & JOHNSON, 1995) UTILIZING THE MILLINGTON AND QUIRK (1961) TORTUOSITY RELATIONSHIP

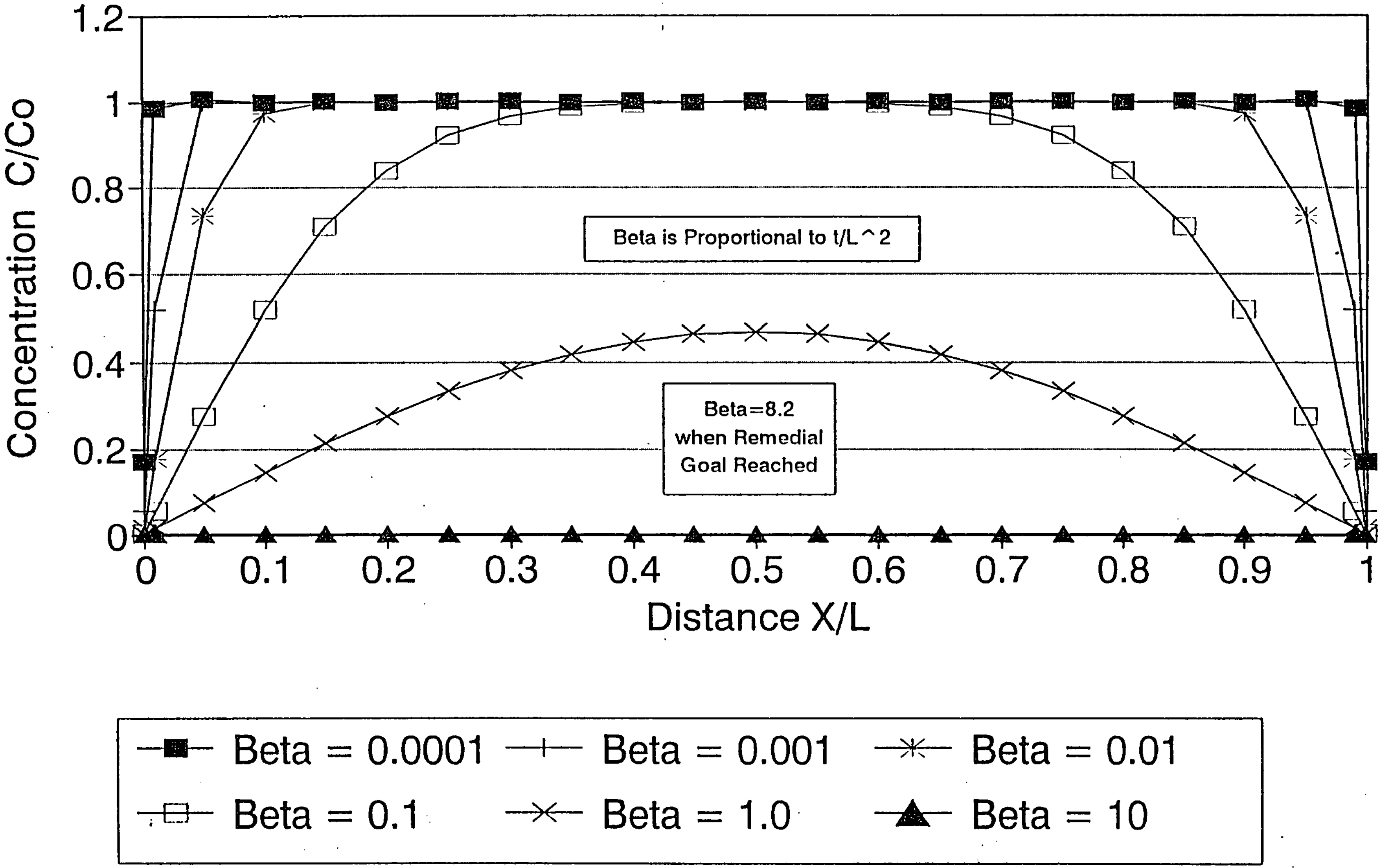
CONTAMINANT TRANSPORT - DIFFUSION

- SCENARIO 1 - NO RESIDUAL TCE IS PRESENT
  - INITIAL CONCENTRATION IS MAXIMUM POSSIBLE WITHOUT RESIDUAL TCE
    - VERY OPTIMISTIC
  - ONE-DIMENSIONAL DIFFUSION TO EDGES OF "CONTROL VOLUME"
  - PREDICT TIME TO REACH CLEANUP CRITERIA
  - SOLUTIONS TO THE DIFFUSION EQUATIONS WELL KNOWN
    (BIRD, STEWART, LIGHTFOOT 1962 - TRANSPORT PHENOMENA)
    (BOYCE & DiPRIMA, 1977 ELEMENTARY DIFFERENTIAL EQUATIONS & B.V. PROBLEMS)



Dimensionless Concentration Profile

Scenario 1 - No Residual TCE, 1-D Diffusion



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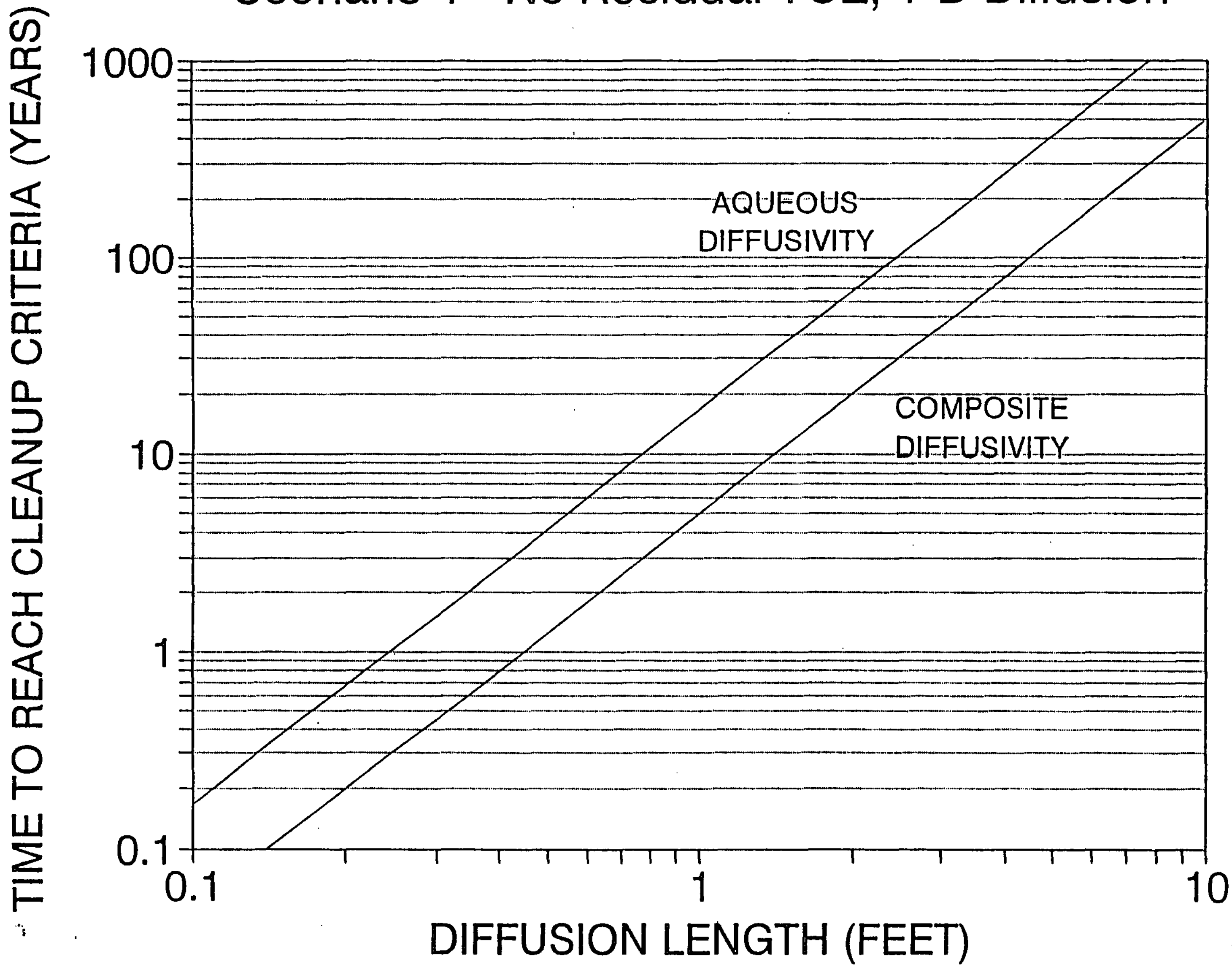
DIFFUSION OF TCE

Scenario 1 - No Residual TCE, 1-D Diffusion



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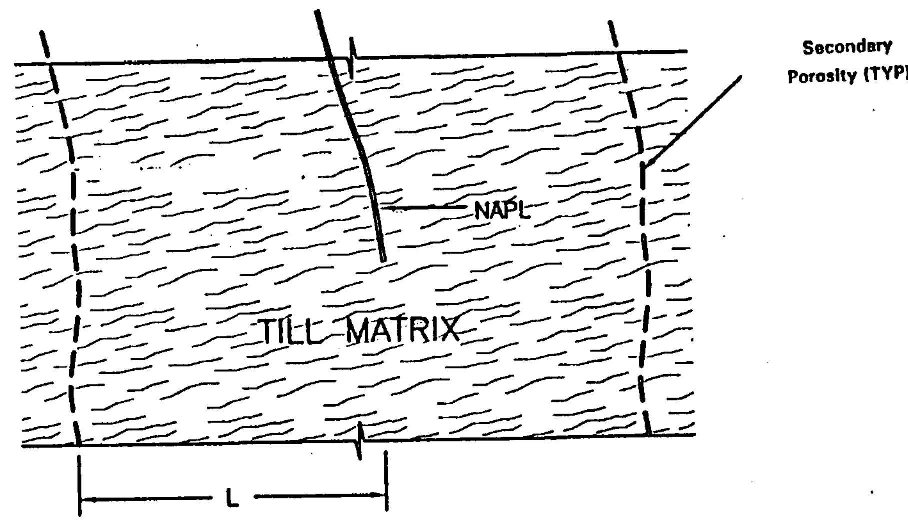
CONTAMINANT TRANSPORT - DIFFUSION

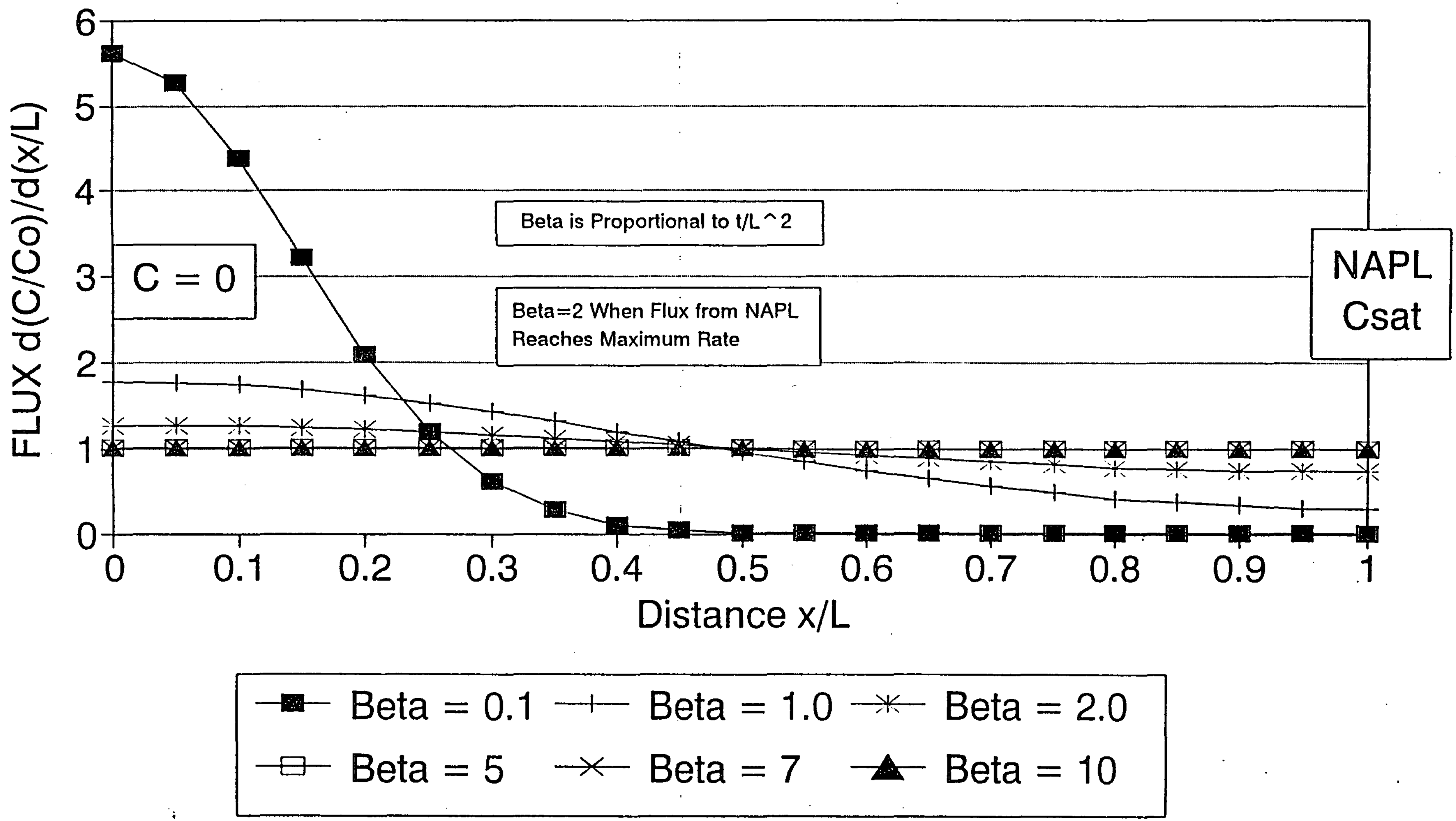
- SCENARIO 2 - RESIDUAL TCE IS PRESENT
  - INITIAL CONCENTRATION IN <u>TILL MATRIX</u> IS MAXIMUM POSSIBLE WITHOUT RESIDUAL TCE
  - ONE-DIMENSIONAL DIFFUSION TO EDGE OF "CONTROL VOLUME"
  - SOLUTION
    - STEADY STATE
    - DEPLETE RESIDUAL TCE



- INITIAL CONDITIONS - EQUIVALENT TO THOSE USED IN ADVECTION MODEL
  - ALL RESIDUAL TCE ASSUMED TO BE IN SECONDARY POROSITY
  - NOT RESIDUAL TCE IN TILL MATRIX

DIFFUSION OF TCE

Scenario 2a - Residual TCE, 1D Diffusion



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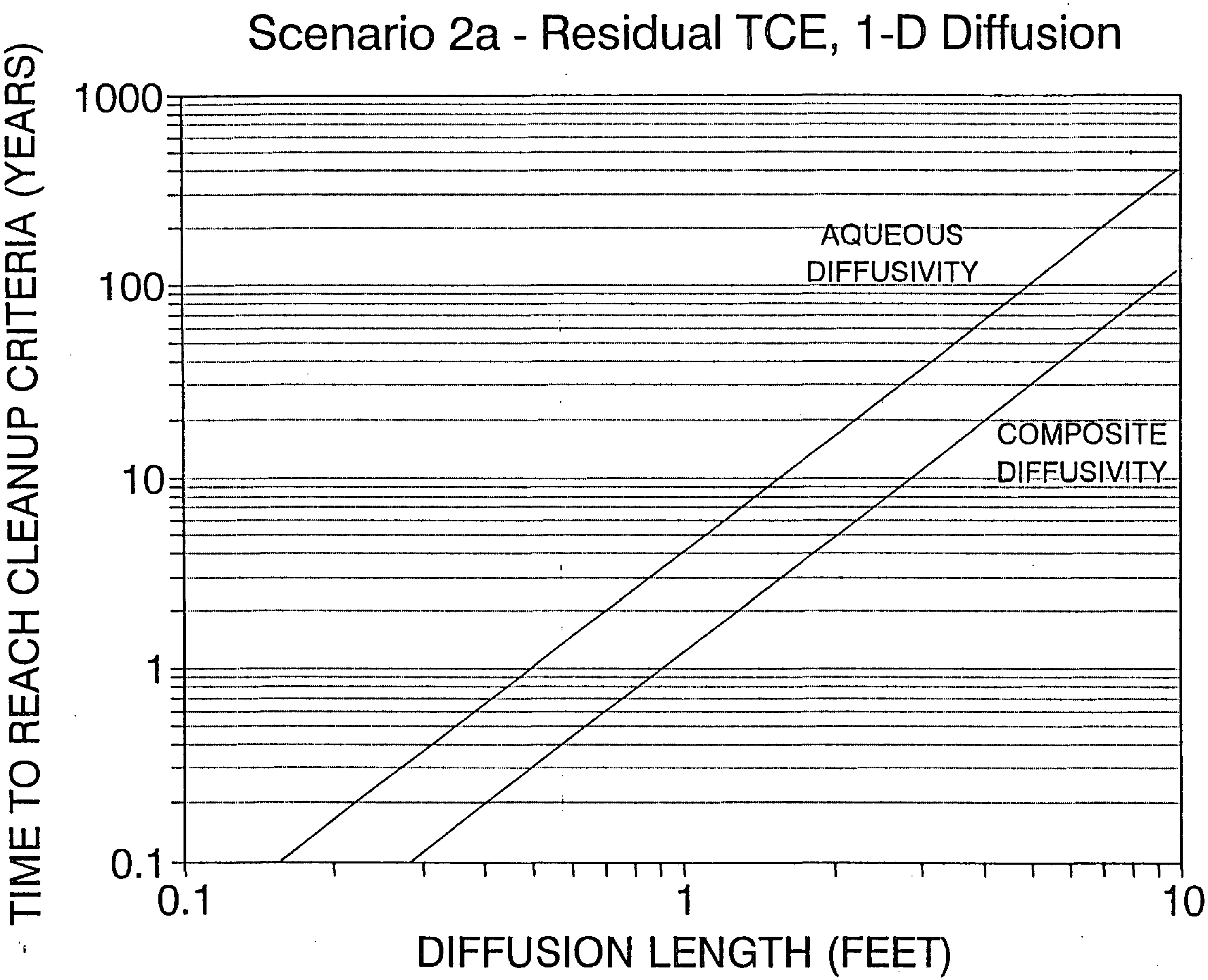
TIME TO REACH STEADY STATE



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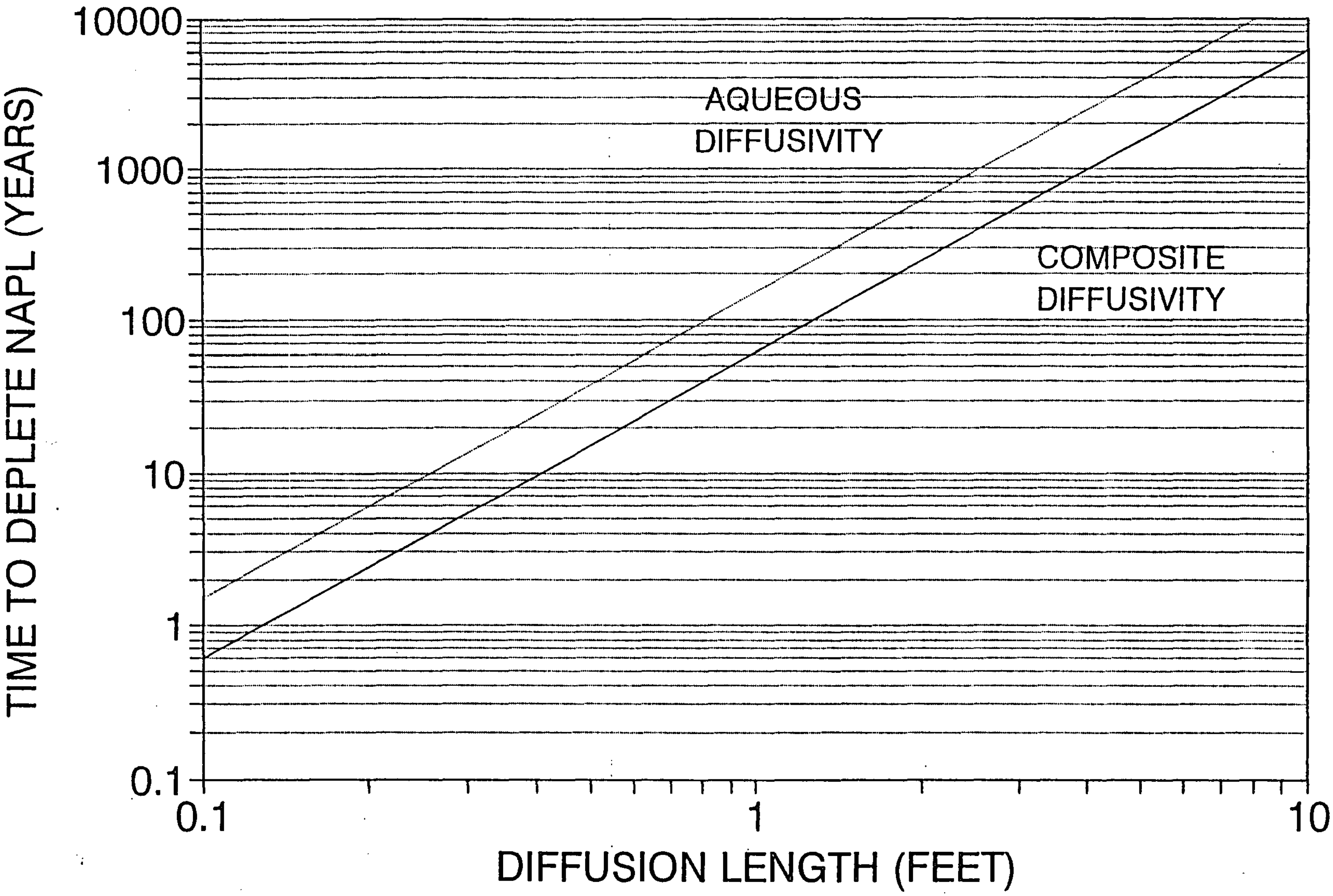
DIFFUSION OF TCE

Scenario 2b - Residual TCE, 1-D Diffusion



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SUMMARY

- CAN DEWATER - SIGNIFICANT DEWATERING IN ONE YEAR
- ADVECTION MODEL
  - RELATED CLEANUP TIME FRAME TO DESIGN CRITERIA
  - MOST DIFFICULT TO REMEDIATE TCE IN LOWER TILL
  - FOR 10 YEAR MINIMUM CLEANUP TIMEFRAME

PRESSURE DIFFERENTIAL	MINIMUM SPACING REQUIREMENTS
1.1 ATM - PRACTICAL	6.5 FEET ($kv = 1.6E\text{-}10\ CM^2$)
1.1 ATM - PRACTICAL	2 FEET ($kh = 1.6E\text{-}11\ CM^2$)
2.4 ATM - POSSIBLE IN LOWER TILL	9 FEET ($kv = 1.6E\text{-}10\ CM^2$)
2.4 ATM - POSSIBLE IN LOWER TILL	3 FEET ($kh = 1.6E\text{-}11\ CM^2$)

- DIFFUSION MODEL
  - CHARACTERIZED A FEW DIFFUSION SCENARIOS, RESIDUAL TCE IS A PROBLEM
  - DIFFUSION LENGTH 1 FOOT
    1 YEAR TO START DEPLETING RESIDUAL TCE
    60 YEARS TO DEPLETE RESIDUAL TCE
    10 YEARS TO REACH CLEANUP CRITERIA

APPENDIX C

APPENDIX C

MODIFIED CONCEPTUAL REMEDIAL DESIGN

146 Hartfc ›ad, Manchester, CT 06040-5921
TEL 860 646-2469 FAX 860 643-6313

1200 Converse Street, Longmeadow, MA 01106-1721
TEL 413 567-9886 FAX 413 567-8936

Providence, RI TEL 401 828-3510

Fuss & O'Neill Inc. *Consulting Engineers*

Solid Waste Management
Industrial/Hazardous Waste Management
Stream Impact Analysis
Water Resources Engineering
Transportation Engineering
Environmental Field Services
Environmental Engineering
Wastewater Management
Site Planning/Engineering
Hydrogeology
Park Design
Surveying

March 11, 1996

Ms. Elise Jakabhazy, RPM
U.S. Environmental Protection Agency
Waste Management Division
JFK Federal Building (HEC-CAN6)
Boston, MA 02203-2211

RE: Linemaster Switch Corporation
Modified Conceptual Remedial Design

Dear Ms. Jakabhazy:

At the close of the meeting on March 6 at Linemaster the Agency requested a proposal for a modification to the Conceptual Remedial Design for the Zone 1 area. This request is the result of the determination by Fuss & O'Neill that remediation of the entire Zone 1 area in accordance with the cleanup levels and duration delineated in the Record of Decision (ROD) is technically infeasible. This conclusion was based on a series of analyses conducted by Fuss & O'Neill, as described below. At the meeting neither EPA nor DEP expressed disagreement with the general concept. The Agencies did request written elaboration of the preliminary remedial concept presented at the meeting and a recommendation for proceeding.

INFEASIBILITY

Briefly, field and laboratory soil data obtained during the Zone 1 delineation borings and the soil fracturing pilot test were summarized in a Technical Memorandum distributed on February 27. This summary included an analysis of hydraulic conductivity and air permeability. It was estimated that dewatering to the lowest fracture could be achieved in one to two years.

A simple advection contaminant transport model was used to estimate the number of soil pore volume changes that would be required to remove residual TCE and TCE partitioned into the soil matrix. The model estimated 3,900 pore volume changes would be required to reach the cleanup concentration of 100 ug/kg. Depending on the bulk versus matrix permeability, 73 to 220 soil pore volume changes per day may be required to remediation the Zone 1 area within the ten year goal. Relating this to the permeability of the overburden, the range of fracture spacing could be approximately one foot to approximately seven feet.

Contaminant transport may be diffusion limited. The time for TCE to diffuse sufficiently to reach the cleanup concentration could approach 70 years. Agency assumptions indicate that the duration may be three times, or more, longer.

Evaluation of the pilot test for the ability to propagate fractures indicates that fracture spacing should be in the range of five to ten feet. The realities of the site specific overburden characteristics and the limitations of the fracturing/DVE remedial technologies have been considered. The conclusion was that remediation of the Zone 1 area to the required cleanup concentrations in a responsive time frame technically, is infeasible.

MODIFIED REMEDIAL APPROACH

To address source control and migration management as referenced in the RI/FS and in the ROD, a modified conceptual approach is discussed below. The concept presented are illustrative. A more comprehensively developed conceptual design would be presented in the Conceptual Design Report.

Groundwater Migration Control

It is envisioned that a series of wells could be installed along a line approximately as shown on the attached sketch. The purpose of the wells would be to control the migration of VOCs in the overburden aquifer. In addition to new wells, existing wells such as FW-A and FW-B may be included. Any other wells that could be used productively could be included. Extracted groundwater would be pumped to the existing Interim Removal Treatment System (IRTS). It is anticipated that the total flow from the overburden groundwater recovery wells would be in the range of one to three gallons per minute. The design flow of the existing IRTS is 120 gpm; it is currently operating at approximately 60 gpm. The design VOC loading is 40,000 ug/l; the current loading is approximately 400-600 ug/l. Consequently, there would be adequate capacity in the existing treatment system.

Source Area DVE

The approximate location of the former dry well is indicated by the 100,000 ug/kg isopleth as shown on the attached figure. Three fractured DVE wells could be installed near the former dry well area as shown in the figure. These wells would have an effective radius of approximately 20 feet. This configuration would address the area within the 10,000 ug/kg isopleth and most of the area within the 1,000 ug/kg isopleth on the east side of the facility.

There are existing monitoring wells located in the vicinity of the potential DVE wells. It is likely that many of them will have to be abandoned so they will not affect the propagation of fractures from the new DVE wells. An abandonment schedule would be prepared as part of the conceptual design.

Source Area Cap

A cap would be placed over the SVE treatment area to reduce infiltration of precipitation, as shown in the attached figure. Cap construction and installation details would be consistent with the description included in the March 1995 Conceptual Design Report.

Monitoring

A monitoring program for both extracted air and groundwater would assess the effectiveness of the proposed remedial system. In addition, a modified program to monitor existing and new monitoring wells would be developed. Likely, this program would be consistent with to the schedule used for the Interim Removal Action (IRA).

Schedule

The schedule for the Remedial Design submitted in December 1995 indicated that the Conceptual Design Report would be submitted on March 29, 1996. As agreed during the March 6 meeting, modifications to the remedial concept will require an adjustment to the schedule. This can be addressed as concurrence on the remedial concept is achieved.

SUMMARY

Field and laboratory measurements of overburden hydraulic conductivity and permeability have been made. The distribution and concentration of TCE in the overburden has been estimated. Combining these factors with the limitations of the DVE/fracturing technologies indicates that it is technically infeasible to remediate the Zone 1 area consistent with the requirements of the ROD. A modified conceptual approach has been presented to address the migration of VOCs in the overburden aquifer and address high concentrations of VOCs in the vicinity of the former dry well. This approach may be more cost effective yet still protect public health and the environment.

Thank you for your consideration of this proposal. We look forward to discussion and development of the details of the design and implementation.

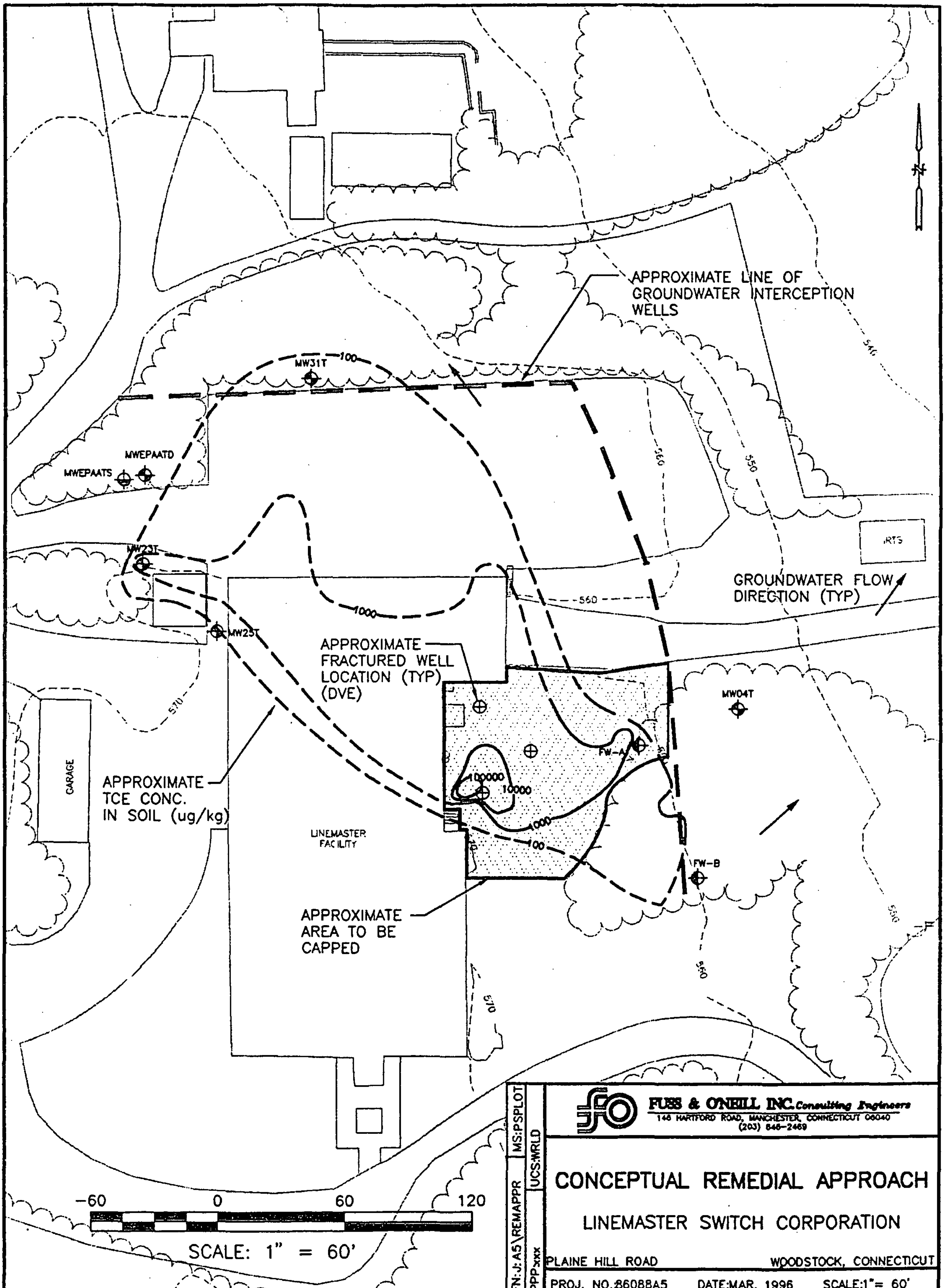
Sincerely,

David L Bramley

David L. Bramley, P.E.
Project Manager

Enclosure

c: w/encl. Gary Kennett - Linemaster
Martin Beskind - DEP
Cinthia McLane - M&E
Larry Murdoch - FRx
Mike Marley - Envirogen
Dominic DiGiulio - EPA



APPROXIMATE LINE OF GROUNDWATER INTERCEPTION WELLS
MW31T
100
MWEPAATD
MWEPAATS
MW23T
MW25T
1000
560
550
570
GROUNDWATER FLOW DIRECTION (TYP)
APPROXIMATE FRACTURED WELL LOCATION (TYP) (DVE)
MW04T
FW-A
FW-B
100000
10000
GARAGE
APPROXIMATE TCE CONC. IN SOIL (ug/kg)
LINEMASTER FACILITY
APPROXIMATE AREA TO BE CAPPED
-60
0
60
120
SCALE: 1" = 60'
FUSS & O'NEILL INC. Consulting Engineers
146 HARTFORD ROAD, MANCHESTER, CONNECTICUT 06040
(203) 646-2469
CONCEPTUAL REMEDIAL APPROACH
LINEMASTER SWITCH CORPORATION
PLAINE HILL ROAD
WOODSTOCK, CONNECTICUT
PROJ. NO.86088A5
DATE:MAR. 1996
SCALE:1"= 60'
FN:J:A5\REMAPPR
MS:P:SPLOT
UCS:WRLD
PPP.xxx

APPENDIX D

APPENDIX D

GEOSTATISTICAL ANALYSIS OF TCE CONCENTRATION AT LINEMASTER (EPA)

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
NATIONAL RISK MANAGEMENT RESEARCH LABORATORY
SUBSURFACE PROTECTION AND REMEDIATION DIVISION
P.O. BOX 1198 • ADA, OK 74820

June 19, 1996

OFFICE OF
RESEARCH AND DEVELOPMENT

MEMORANDUM

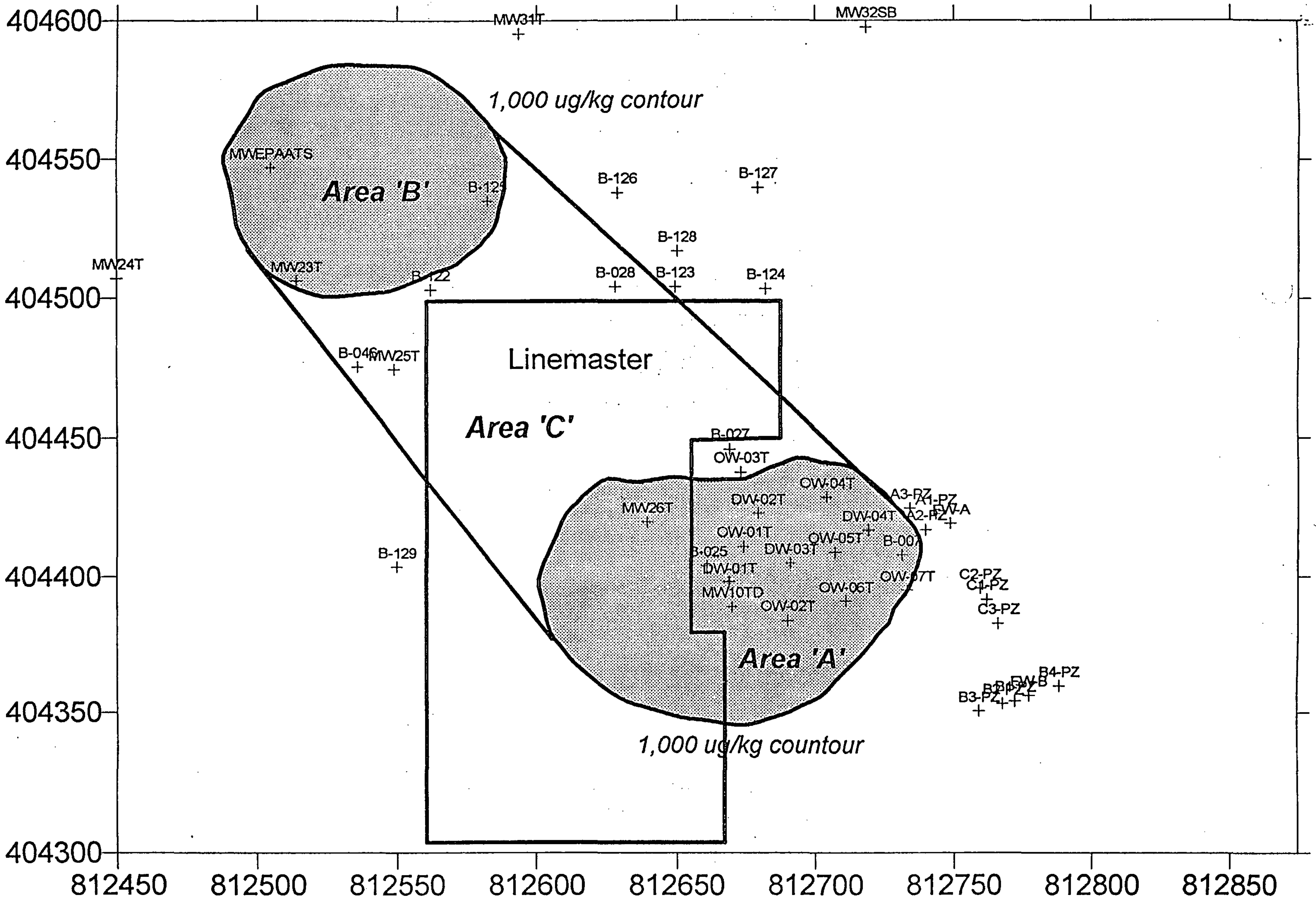
SUBJECT: Geostatistical Analysis of TCE Concentration Data at the Linemaster Switch Site, Woodstock, CT (94-R01-006)

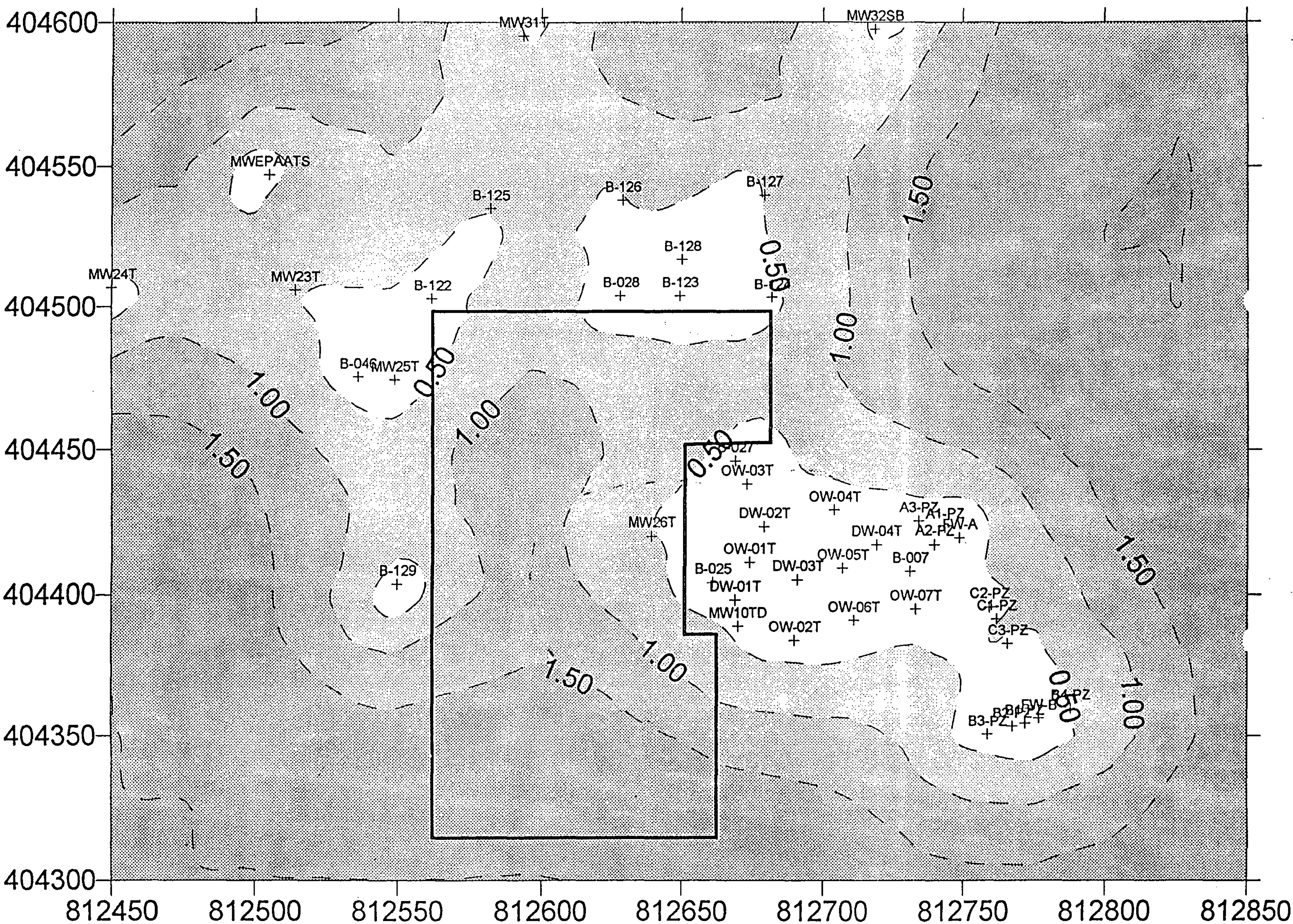
FROM: Dominic C. DiGiulio, Environmental Engineer
Technical Assistance and Technology Transfer Branch

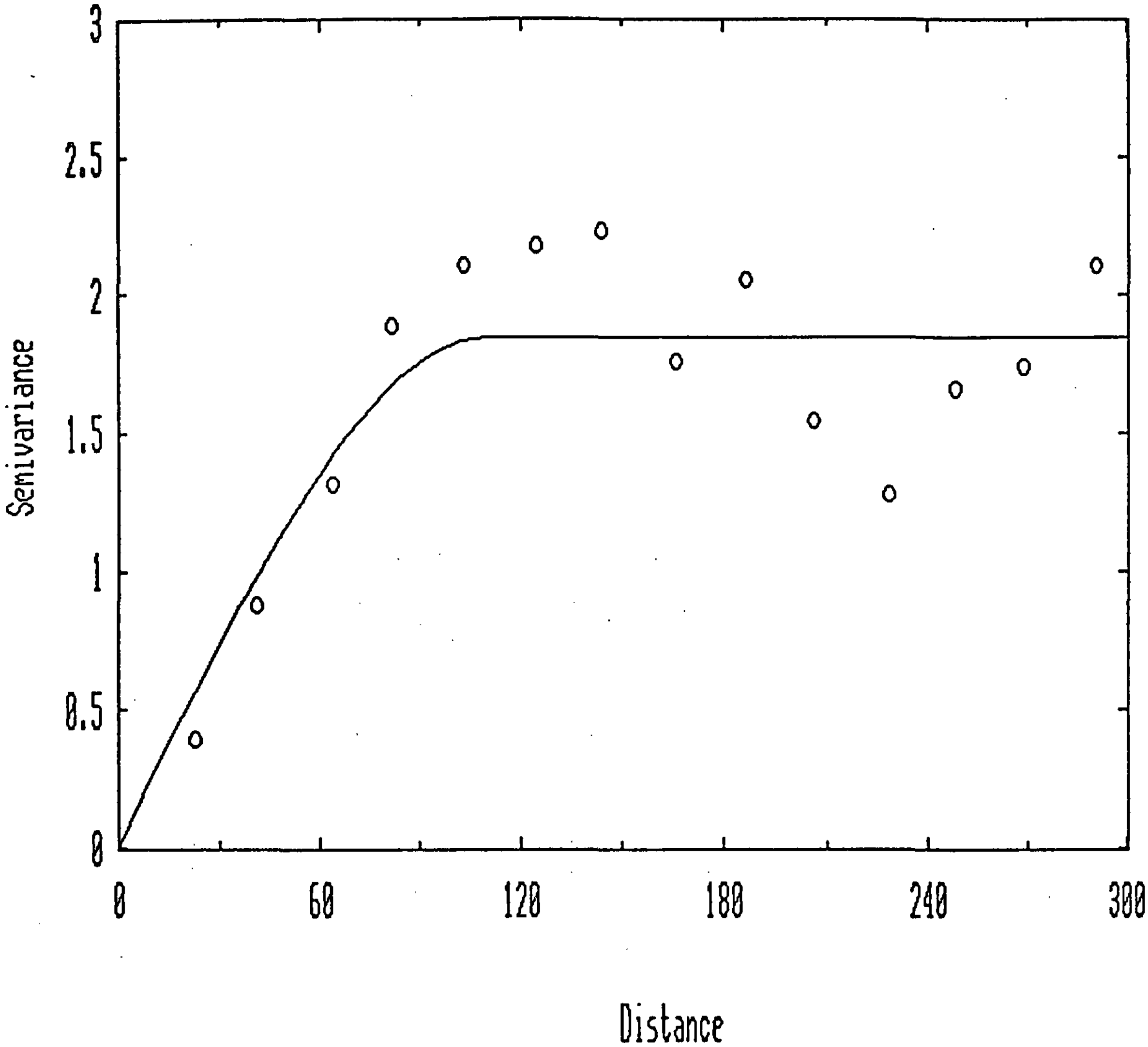
TO: Elise Jakabhazy, Remedial Project Manager
U.S. EPA, Region I

A summary of the geostatistical analysis conducted at the Linemaster Switch Site by Dr. Varadhan Ravi, Dynamac Corp., and myself is attached to formally document our efforts. As stated during conference calls, we eliminated soil concentration data from boreholes having less than four measurements. We also normalized ground-water concentration data specifically at wells MW-10D and MWEPAATS. Our approach was essentially to do kriging with GEOPACK, an EPA-Kerr Research Center software package and contouring using linear interpolation with SURFER. We did not use SURFER to do kriging because the package does not generate variance information which is necessary to evaluate where additional data is necessary. The most recent soils concentration data collected this past week was not included in this analysis.

cc: David Bramley, Fuss&O'Neill Inc. with GEOPACK software, code documentation, and SURFER input files
Anna Krasko, Region I, w/o software
Dick Willey, Region I, w/o software
Ruth Bleyler, Region I, w/o software
Rich Steimle, 51102W, w/o software


1,000 ug/kg contour
Area 'B'
MWEPAATS
B-126
B-127
B-128
MW24T
MW23T
B-122
B-028
B-123
B-124
Linemaster
B-046
MW25T
Area 'C'
B-027
OW-03T
OW-04T
DW-02T
MW26T
A3-PZ
A1-PZ
EW-A
A2-PZ
DW-04T
OW-01T
OW-05T
B-007
B-025
DW-03T
DW-01T
OW-07T
B-129
C2-PZ
C1-PZ
MW10TD
OW-06T
OW-02T
C3-PZ
Area 'A'
B4-PZ
EW-B
B3-PZ
1,000 ug/kg countour
MW31T
MW32SB
404600
404550
404500
404450
404400
404350
404300
812450
812500
812550
812600
812650
812700
812750
812800
812850


404600
404550
404500
404450
404400
404350
404300
812450
812500
812550
812600
812650
812700
812750
812800
812850
MW31T
MW32SB
MWEPAATS
B-125
B-126
B-127
B-128
MW24T
MW23T
B-122
B-028
B-123
B-046
MW25T
0.50
1.00
1.50
OW-03T
OW-04T
DW-02T
A3-PZ
A1-PZ
FW-A
A2-PZ
DW-04T
MW26T
OW-01T
OW-05T
B-007
B-025
DW-03T
DW-01T
B-129
OW-07T
C2-PZ
C1-PZ
OW-06T
MW10TD
OW-02T
C3-PZ
B4-PZ
B3-PZ



Semivariance
3
2.5
2
1.5
1
0.5
0
0
60
120
180
240
300
Distance

1***
*   *
*   NON-LINEAR LEAST SQUARES ANALYSIS   *
*   *
*   Fit A Variogram Model To The Sample Variogram   *
*   Using A Non-Linear Least-Squares Analysis   *
*   *
****

INPUT PARAMETERS
================
MODEL NUMBER.................................... 1
NUMBER OF COEFFICIENTS.......................... 3
MAXIMUM NUMBER OF ITERATIONS.................... 20
RATIO OF COEFFICIENTS CRITERION................. .0005

OBSERVED DATA
=============

OBS. No.	No. COUPLES	DISTANCE	GAMMA
1	73	.2273E+02	.3831E+00
2	103	.4107E+02	.8607E+00
3	94	.6383E+02	.1304E+01
4	85	.8127E+02	.1872E+01
5	96	.1032E+03	.2094E+01
6	88	.1245E+03	.2167E+01
7	77	.1441E+03	.2224E+01
8	54	.1661E+03	.1745E+01
9	62	.1863E+03	.2042E+01
10	74	.2063E+03	.1537E+01
11	53	.2283E+03	.1268E+01
12	42	.2481E+03	.1649E+01
13	39	.2685E+03	.1730E+01
14	27	.2906E+03	.2102E+01

ITER NO	NUGGET	SILL-NUGGET	RANGE	SSQ	MODEL
0	.4449E+00	.2002E+01	.1453E+03	4.850E+00	SPHERIC
1	.1505E+00	.1746E+01	.1167E+03	1.092E+00	SPHERIC
2	.8676E-01	.1765E+01	.1140E+03	1.043E+00	SPHERIC

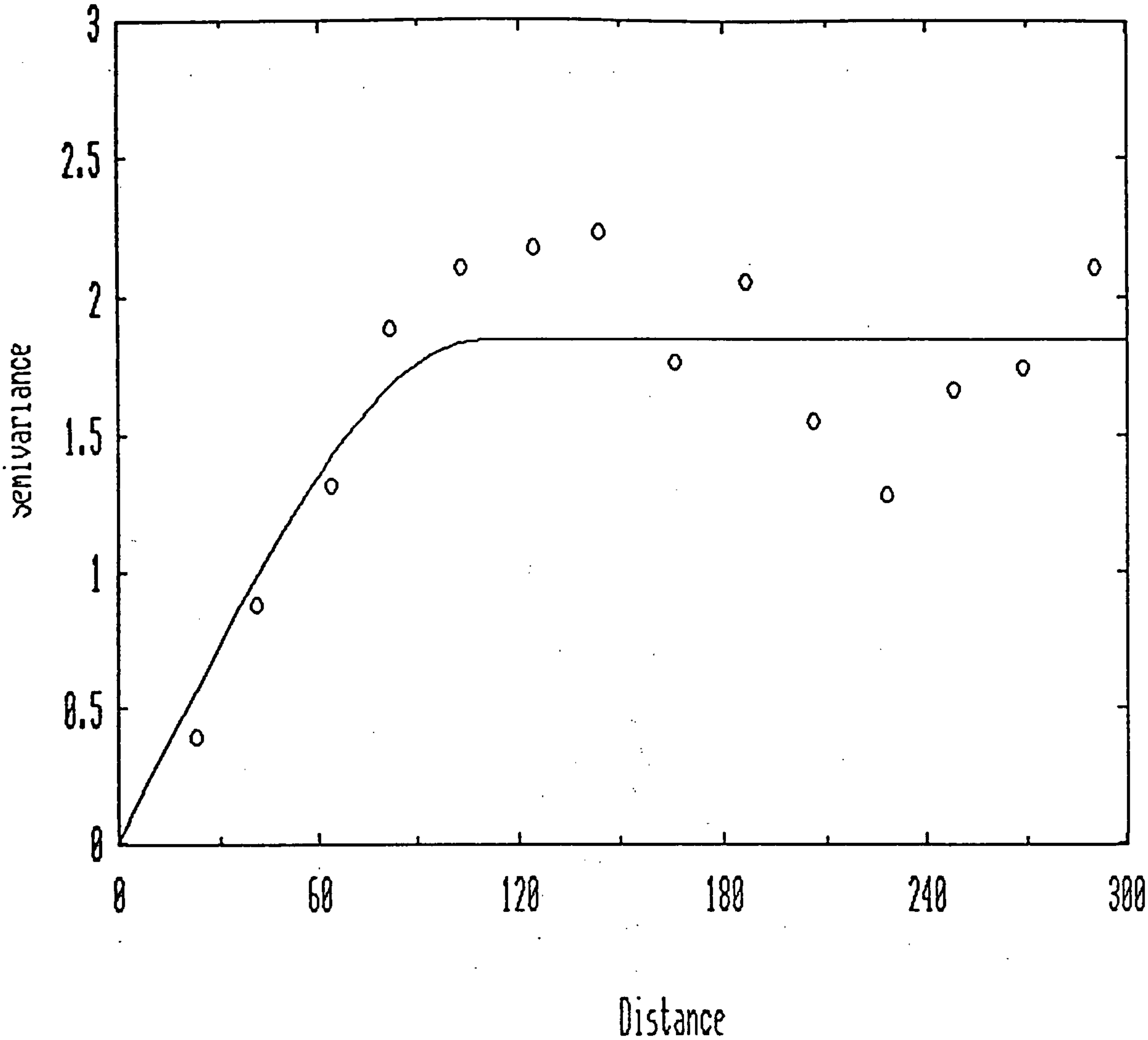
3	.4639E-01	.1801E+01	.1128E+03	1.027E+00	SPHERIC
4	.1941E-01	.1828E+01	.1118E+03	1.017E+00	SPHERIC
5	.6290E-02	.1841E+01	.1113E+03	1.012E+00	SPHERIC
6	.2719E-02	.1845E+01	.1112E+03	1.011E+00	SPHERIC
7	.1225E-02	.1846E+01	.1111E+03	1.010E+00	SPHERIC
8	.4897E-03	.1847E+01	.1111E+03	1.010E+00	SPHERIC
9	.0000E+00	.1847E+01	.1111E+03	1.010E+00	SPHERIC

CORRELATION MATRIX
================

	1	2
1	1.0000	
2	.4326	1.0000

NONLINEAR LEAST-SQUARES ANALYSIS: FINAL RESULTS
==

				95% CONFIDENCE LIMITS	
VARIABLE	VALUE	S.E.COEFF.	T-VALUE	LOWER	UPPER
RANGE	.11107E+03	.19948E+02	.5568E+01	.6760E+02	.1545E+03
SILL-N	.18474E+01	.93826E-01	.1969E+02	.1643E+01	.2052E+01
NUGGET	.00000E+00	--	--	--	--



3
2.5
2
1.5
1
0.5
0
Semivariance
0
60
120
180
240
300
Distance

```
******************************************************************************
*                                                                   *
*                  LINEAR ESTIMATOR CALCULATION                              *
*                ***********************************                 *
*                  DATE: 05/22/1996        TIME: 14:45 pm            *
*                                                                   *
*  Maximum total concentrations of TCE expressed in terms of soil concent  *
*  Linemaster Switch site                                              *
*  May 08, 1996                                                        *
*                                                                   *
*                                                                   *
*                      *****  KRIGING  *****                           *
******************************************************************************
```

INPUT PARAMETERS
===============

NUMBER OF RANDOM FUNCTIONS...................... 1
NUMBER OF DATA POINT READ....................... 47
NUMBER OF NEAREST NEIGHBORS FOR lConcn........... 46
MAXIMUM ALLOWED VALUE FOR lConcn...........**********
MAXIMUM ALLOWED RADIUS.......................... 999.990

COVARIANCE/VARIOGRAM COEFFICIENTS
================================

VARIOGRAMS	MODEL	NUGGET	SILL-NUGGET	RANGE
lConcn/lConcn	SPHER	.0000	1.8500	111.0000

SITE	X	Y	lConcn
1	7.4330E+02	4.2300E+02	3.98500E+00
2	7.3960E+02	4.1710E+02	3.58600E+00
3	7.3400E+02	4.2510E+02	2.46200E+00
4	7.3100E+02	4.0800E+02	3.60400E+00
5	6.6100E+02	4.0400E+02	3.47700E+00
6	6.6900E+02	4.4600E+02	1.00000E+00
7	6.2800E+02	5.0400E+02	1.11400E+00
8	5.3600E+02	4.7500E+02	6.99000E-01
9	5.6210E+02	5.0280E+02	3.26100E+00
10	6.4920E+02	5.0400E+02	2.53900E+00

11	6.8180E+02	5.0330E+02	3.11800E+00
12	5.8250E+02	5.3500E+02	3.27700E+00
13	6.2880E+02	5.3800E+02	2.35400E+00
14	6.7910E+02	5.3980E+02	2.42800E+00
15	6.5000E+02	5.1700E+02	2.20400E+00
16	5.5000E+02	4.0350E+02	1.00000E+00
17	9.3280E+02	5.0300E+02	6.99000E-01
18	7.7180E+02	3.5440E+02	6.99000E-01
19	7.6740E+02	3.5340E+02	1.83300E+00
20	7.5860E+02	3.5060E+02	2.38700E+00

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21	7.8780E+02	3.5980E+02	1.79200E+00
22	7.6190E+02	3.9160E+02	1.64300E+00
23	7.5940E+02	3.9570E+02	2.27200E+00
24	7.6580E+02	3.8290E+02	2.16100E+00
25	6.6900E+02	3.9800E+02	5.32200E+00
26	6.7920E+02	4.2310E+02	3.99100E+00
27	6.9100E+02	4.0500E+02	3.99600E+00
28	7.1900E+02	4.1700E+02	3.88600E+00
29	7.4860E+02	4.1930E+02	2.15800E+00
30	7.7680E+02	3.5630E+02	1.83900E+00
31	6.7000E+02	3.8900E+02	4.76000E+00
32	5.1400E+02	5.0600E+02	3.56800E+00
33	4.5000E+02	5.0700E+02	6.99000E-01
34	5.4900E+02	4.7400E+02	6.99000E-01
35	6.3940E+02	4.2000E+02	4.63500E+00
36	5.9360E+02	5.9540E+02	2.43300E+00
37	7.1840E+02	5.9760E+02	6.99000E-01
38	9.0750E+02	6.5370E+02	1.74000E+00
39	4.8540E+02	7.4080E+02	6.99000E-01
40	6.7400E+02	4.1100E+02	4.38000E+00
41	6.9000E+02	3.8400E+02	3.95900E+00
42	6.7300E+02	4.3800E+02	3.78500E+00
43	7.0400E+02	4.2900E+02	3.36200E+00
44	7.0700E+02	4.0900E+02	3.90300E+00
45	7.1100E+02	3.9100E+02	3.30100E+00
46	7.3300E+02	3.9500E+02	2.83300E+00
47	5.0500E+02	5.4700E+02	3.73200E+00

ESTIMATED VALUES
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X0	Y0	lConcn	VARIANCE	N1
4.0000E+02	3.0000E+02	1.9542E+00	2.0020E+00	46
4.0000E+02	3.2500E+02	1.9542E+00	2.0020E+00	46

4.0000E+02	3.5000E+02	1.9542E+00	2.0020E+00	46
4.0000E+02	3.7500E+02	1.9542E+00	2.0020E+00	46
4.0000E+02	4.0000E+02	1.9542E+00	2.0020E+00	46
4.0000E+02	4.2500E+02	1.9053E+00	1.9941E+00	46
4.0000E+02	4.5000E+02	1.7009E+00	1.9334E+00	46
4.0000E+02	4.7500E+02	1.4386E+00	1.7898E+00	46
4.0000E+02	5.0000E+02	1.3279E+00	1.6573E+00	46
4.0000E+02	5.2500E+02	1.3842E+00	1.7017E+00	46
4.0000E+02	5.5000E+02	1.6266E+00	1.8609E+00	46
4.0000E+02	5.7500E+02	1.8857E+00	1.9698E+00	46
4.0000E+02	6.0000E+02	2.0329E+00	2.0010E+00	46
4.0000E+02	6.2500E+02	2.0397E+00	2.0020E+00	46
4.0000E+02	6.5000E+02	2.0397E+00	2.0020E+00	46
4.0000E+02	6.7500E+02	2.0381E+00	2.0016E+00	46
4.0000E+02	7.0000E+02	1.9982E+00	1.9909E+00	46
4.0000E+02	7.2500E+02	1.9514E+00	1.9746E+00	46
4.0000E+02	7.5000E+02	1.9446E+00	1.9718E+00	46
4.0000E+02	7.7500E+02	1.9841E+00	1.9864E+00	46
4.2500E+02	3.0000E+02	1.9542E+00	2.0020E+00	46
4.2500E+02	3.2500E+02	1.9542E+00	2.0020E+00	46
4.2500E+02	3.5000E+02	1.9542E+00	2.0020E+00	46
4.2500E+02	3.7500E+02	1.9542E+00	2.0020E+00	46
4.2500E+02	4.0000E+02	1.9539E+00	2.0019E+00	46

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4.2500E+02	4.2500E+02	1.8191E+00	1.9739E+00	46
4.2500E+02	4.5000E+02	1.4994E+00	1.8244E+00	46
4.2500E+02	4.7500E+02	1.1378E+00	1.4634E+00	46
4.2500E+02	5.0000E+02	9.2228E-01	1.0537E+00	46
4.2500E+02	5.2500E+02	1.0671E+00	1.2049E+00	46
4.2500E+02	5.5000E+02	1.4463E+00	1.6283E+00	46
4.2500E+02	5.7500E+02	1.7978E+00	1.8907E+00	46
4.2500E+02	6.0000E+02	2.0146E+00	1.9879E+00	46
4.2500E+02	6.2500E+02	2.0397E+00	2.0020E+00	46
4.2500E+02	6.5000E+02	2.0391E+00	2.0018E+00	46
4.2500E+02	6.7500E+02	1.9680E+00	1.9808E+00	46
4.2500E+02	7.0000E+02	1.8298E+00	1.9127E+00	46
4.2500E+02	7.2500E+02	1.7109E+00	1.8252E+00	46
4.2500E+02	7.5000E+02	1.6941E+00	1.8108E+00	46
4.2500E+02	7.7500E+02	1.7926E+00	1.8883E+00	46
4.5000E+02	3.0000E+02	1.9542E+00	2.0020E+00	46
4.5000E+02	3.2500E+02	1.9542E+00	2.0020E+00	46
4.5000E+02	3.5000E+02	1.9542E+00	2.0020E+00	46
4.5000E+02	3.7500E+02	1.9509E+00	2.0003E+00	46
4.5000E+02	4.0000E+02	1.9426E+00	1.9973E+00	46
4.5000E+02	4.2500E+02	1.7532E+00	1.9570E+00	46
4.5000E+02	4.5000E+02	1.4337E+00	1.7515E+00	46

4.5000E+02	4.7500E+02	1.1193E+00	1.2351E+00	46
4.5000E+02	5.0000E+02	8.1612E-01	3.3184E-01	46
4.5000E+02	5.2500E+02	1.2575E+00	7.5418E-01	46
4.5000E+02	5.5000E+02	1.7398E+00	1.3847E+00	46
4.5000E+02	5.7500E+02	1.9639E+00	1.7297E+00	46
4.5000E+02	6.0000E+02	2.0859E+00	1.9263E+00	46
4.5000E+02	6.2500E+02	2.0653E+00	1.9945E+00	46
4.5000E+02	6.5000E+02	2.0110E+00	1.9947E+00	46
4.5000E+02	6.7500E+02	1.8483E+00	1.9239E+00	46
4.5000E+02	7.0000E+02	1.6004E+00	1.7201E+00	46
4.5000E+02	7.2500E+02	1.3728E+00	1.4306E+00	46
4.5000E+02	7.5000E+02	1.3377E+00	1.3773E+00	46
4.5000E+02	7.7500E+02	1.5323E+00	1.6438E+00	46
4.7500E+02	3.0000E+02	1.9542E+00	2.0020E+00	46
4.7500E+02	3.2500E+02	1.9538E+00	2.0018E+00	46
4.7500E+02	3.5000E+02	1.9310E+00	1.9879E+00	46
4.7500E+02	3.7500E+02	1.8948E+00	1.9539E+00	46
4.7500E+02	4.0000E+02	1.8101E+00	1.9252E+00	46
4.7500E+02	4.2500E+02	1.5140E+00	1.8777E+00	46
4.7500E+02	4.5000E+02	1.2525E+00	1.6668E+00	46
4.7500E+02	4.7500E+02	1.3118E+00	1.2436E+00	46
4.7500E+02	5.0000E+02	1.6435E+00	8.0901E-01	46
4.7500E+02	5.2500E+02	2.1633E+00	8.3018E-01	46
4.7500E+02	5.5000E+02	2.4970E+00	1.0474E+00	46
4.7500E+02	5.7500E+02	2.4166E+00	1.4381E+00	46
4.7500E+02	6.0000E+02	2.2506E+00	1.7928E+00	46
4.7500E+02	6.2500E+02	2.1162E+00	1.9686E+00	46
4.7500E+02	6.5000E+02	1.9822E+00	1.9850E+00	46
4.7500E+02	6.7500E+02	1.7610E+00	1.8654E+00	46
4.7500E+02	7.0000E+02	1.4253E+00	1.5060E+00	46
4.7500E+02	7.2500E+02	1.0384E+00	8.2781E-01	46
4.7500E+02	7.5000E+02	9.4926E-01	6.3148E-01	46
4.7500E+02	7.7500E+02	1.3243E+00	1.3562E+00	46
5.0000E+02	3.0000E+02	1.9542E+00	2.0020E+00	46
5.0000E+02	3.2500E+02	1.9332E+00	1.9895E+00	46
5.0000E+02	3.5000E+02	1.8668E+00	1.9172E+00	46

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5.0000E+02	3.7500E+02	1.7821E+00	1.7673E+00	46
5.0000E+02	4.0000E+02	1.4573E+00	1.6508E+00	46
5.0000E+02	4.2500E+02	1.0883E+00	1.5988E+00	46
5.0000E+02	4.5000E+02	9.9190E-01	1.4044E+00	46
5.0000E+02	4.7500E+02	1.5031E+00	1.0272E+00	46
5.0000E+02	5.0000E+02	2.6507E+00	5.9123E-01	46
5.0000E+02	5.2500E+02	3.3335E+00	5.9311E-01	46
5.0000E+02	5.5000E+02	3.4985E+00	2.7638E-01	46
5.0000E+02	5.7500E+02	2.9673E+00	1.1283E+00	46

5.0000E+02	6.0000E+02	2.4174E+00	1.6791E+00	46
5.0000E+02	6.2500E+02	2.1510E+00	1.9399E+00	46
5.0000E+02	6.5000E+02	1.9908E+00	1.9846E+00	46
5.0000E+02	6.7500E+02	1.7701E+00	1.8721E+00	46
5.0000E+02	7.0000E+02	1.4442E+00	1.5320E+00	46
5.0000E+02	7.2500E+02	1.0839E+00	9.2223E-01	46
5.0000E+02	7.5000E+02	1.0091E+00	7.6503E-01	46
5.0000E+02	7.7500E+02	1.3475E+00	1.3924E+00	46
5.2500E+02	3.0000E+02	1.9528E+00	2.0013E+00	46
5.2500E+02	3.2500E+02	1.9025E+00	1.9623E+00	46
5.2500E+02	3.5000E+02	1.7968E+00	1.7865E+00	46
5.2500E+02	3.7500E+02	1.6204E+00	1.3977E+00	46
5.2500E+02	4.0000E+02	1.0749E+00	1.0415E+00	46
5.2500E+02	4.2500E+02	6.7723E-01	1.1072E+00	46
5.2500E+02	4.5000E+02	5.9767E-01	9.6141E-01	46
5.2500E+02	4.7500E+02	1.1611E+00	4.5140E-01	46
5.2500E+02	5.0000E+02	3.0377E+00	4.2165E-01	46
5.2500E+02	5.2500E+02	3.9579E+00	6.3793E-01	46
5.2500E+02	5.5000E+02	3.9394E+00	7.5027E-01	46
5.2500E+02	5.7500E+02	3.3059E+00	1.2149E+00	46
5.2500E+02	6.0000E+02	2.4850E+00	1.6398E+00	46
5.2500E+02	6.2500E+02	2.1615E+00	1.8815E+00	46
5.2500E+02	6.5000E+02	2.0328E+00	1.9720E+00	46
5.2500E+02	6.7500E+02	1.8692E+00	1.9344E+00	46
5.2500E+02	7.0000E+02	1.6390E+00	1.7595E+00	46
5.2500E+02	7.2500E+02	1.4335E+00	1.5173E+00	46
5.2500E+02	7.5000E+02	1.4026E+00	1.4741E+00	46
5.2500E+02	7.7500E+02	1.5114E+00	1.6901E+00	46
5.5000E+02	3.0000E+02	1.9504E+00	2.0001E+00	46
5.5000E+02	3.2500E+02	1.8883E+00	1.9462E+00	46
5.5000E+02	3.5000E+02	1.7652E+00	1.7091E+00	46
5.5000E+02	3.7500E+02	1.6135E+00	1.1320E+00	46
5.5000E+02	4.0000E+02	1.0950E+00	1.7039E-01	46
5.5000E+02	4.2500E+02	7.9487E-01	7.6279E-01	46
5.5000E+02	4.5000E+02	5.7562E-01	7.9400E-01	46
5.5000E+02	4.7500E+02	7.9895E-01	6.7790E-02	46
5.5000E+02	5.0000E+02	2.8941E+00	4.1602E-01	46
5.5000E+02	5.2500E+02	3.8261E+00	7.1831E-01	46
5.5000E+02	5.5000E+02	3.8413E+00	9.8043E-01	46
5.5000E+02	5.7500E+02	3.2965E+00	1.2345E+00	46
5.5000E+02	6.0000E+02	2.4870E+00	1.4272E+00	46
5.5000E+02	6.2500E+02	2.1693E+00	1.6775E+00	46
5.5000E+02	6.5000E+02	2.0734E+00	1.8901E+00	46
5.5000E+02	6.7500E+02	1.9941E+00	1.9706E+00	46
5.5000E+02	7.0000E+02	1.8647E+00	1.9333E+00	46
5.5000E+02	7.2500E+02	1.7597E+00	1.8644E+00	46

5.5000E+02	7.5000E+02	1.6672E+00	1.8468E+00	46
5.5000E+02	7.7500E+02	1.7476E+00	1.9066E+00	46
5.7500E+02	3.0000E+02	1.9528E+00	2.0013E+00	46

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5.7500E+02	3.2500E+02	1.9025E+00	1.9623E+00	46
5.7500E+02	3.5000E+02	1.8765E+00	1.7814E+00	46
5.7500E+02	3.7500E+02	2.0102E+00	1.3594E+00	46
5.7500E+02	4.0000E+02	1.8758E+00	9.5491E-01	46
5.7500E+02	4.2500E+02	1.4422E+00	1.0423E+00	46
5.7500E+02	4.5000E+02	1.1193E+00	1.0708E+00	46
5.7500E+02	4.7500E+02	1.4517E+00	8.2003E-01	46
5.7500E+02	5.0000E+02	2.6243E+00	5.0148E-01	46
5.7500E+02	5.2500E+02	3.3187E+00	4.2418E-01	46
5.7500E+02	5.5000E+02	3.3581E+00	6.4467E-01	46
5.7500E+02	5.7500E+02	2.9544E+00	8.7936E-01	46
5.7500E+02	6.0000E+02	2.4634E+00	8.1405E-01	46
5.7500E+02	6.2500E+02	2.1836E+00	1.3046E+00	46
5.7500E+02	6.5000E+02	2.0940E+00	1.7589E+00	46
5.7500E+02	6.7500E+02	2.0575E+00	1.9590E+00	46
5.7500E+02	7.0000E+02	2.0155E+00	1.9950E+00	46
5.7500E+02	7.2500E+02	1.8830E+00	1.9749E+00	46
5.7500E+02	7.5000E+02	1.8779E+00	1.9731E+00	46
5.7500E+02	7.7500E+02	1.9071E+00	1.9828E+00	46
6.0000E+02	3.0000E+02	1.9542E+00	2.0020E+00	46
6.0000E+02	3.2500E+02	1.9535E+00	1.9791E+00	46
6.0000E+02	3.5000E+02	2.2415E+00	1.8336E+00	46
6.0000E+02	3.7500E+02	2.7838E+00	1.5268E+00	46
6.0000E+02	4.0000E+02	2.9744E+00	1.2200E+00	46
6.0000E+02	4.2500E+02	2.4478E+00	1.1538E+00	46
6.0000E+02	4.5000E+02	1.6580E+00	1.2028E+00	46
6.0000E+02	4.7500E+02	1.4045E+00	1.0592E+00	46
6.0000E+02	5.0000E+02	1.7667E+00	7.6840E-01	46
6.0000E+02	5.2500E+02	2.5240E+00	5.7430E-01	46
6.0000E+02	5.5000E+02	2.7318E+00	6.5722E-01	46
6.0000E+02	5.7500E+02	2.4859E+00	6.9832E-01	46
6.0000E+02	6.0000E+02	2.3422E+00	3.7163E-01	46
6.0000E+02	6.2500E+02	2.1931E+00	1.1760E+00	46
6.0000E+02	6.5000E+02	2.0991E+00	1.7192E+00	46
6.0000E+02	6.7500E+02	2.0597E+00	1.9511E+00	46
6.0000E+02	7.0000E+02	1.9412E+00	1.9903E+00	46
6.0000E+02	7.2500E+02	1.9399E+00	1.9915E+00	46
6.0000E+02	7.5000E+02	1.9399E+00	1.9915E+00	46
6.0000E+02	7.7500E+02	1.9399E+00	1.9915E+00	46
6.2500E+02	3.0000E+02	2.0464E+00	1.9982E+00	46
6.2500E+02	3.2500E+02	2.2051E+00	1.9267E+00	46
6.2500E+02	3.5000E+02	2.7813E+00	1.7111E+00	46

6.2500E+02	3.7500E+02	3.4386E+00	1.3496E+00	46
6.2500E+02	4.0000E+02	3.9103E+00	8.9796E-01	46
6.2500E+02	4.2500E+02	3.6391E+00	6.3808E-01	46
6.2500E+02	4.5000E+02	2.2330E+00	9.7787E-01	46
6.2500E+02	4.7500E+02	1.4639E+00	8.9906E-01	46
6.2500E+02	5.0000E+02	1.1871E+00	2.3190E-01	46
6.2500E+02	5.2500E+02	1.9525E+00	4.0180E-01	46
6.2500E+02	5.5000E+02	2.4370E+00	5.0972E-01	46
6.2500E+02	5.7500E+02	2.1751E+00	9.8915E-01	46
6.2500E+02	6.0000E+02	2.1138E+00	1.1718E+00	46
6.2500E+02	6.2500E+02	2.1153E+00	1.5043E+00	46
6.2500E+02	6.5000E+02	2.0822E+00	1.8245E+00	46
6.2500E+02	6.7500E+02	1.9598E+00	1.9631E+00	46
6.2500E+02	7.0000E+02	1.9400E+00	1.9914E+00	46
6.2500E+02	7.2500E+02	1.9399E+00	1.9915E+00	46
6.2500E+02	7.5000E+02	1.9399E+00	1.9915E+00	46
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6.2500E+02	7.7500E+02	1.9399E+00	1.9915E+00	46
6.5000E+02	3.0000E+02	2.0987E+00	1.9801E+00	46
6.5000E+02	3.2500E+02	2.2593E+00	1.8233E+00	46
6.5000E+02	3.5000E+02	3.0407E+00	1.4657E+00	46
6.5000E+02	3.7500E+02	3.8978E+00	9.1885E-01	46
6.5000E+02	4.0000E+02	3.9960E+00	4.2348E-01	46
6.5000E+02	4.2500E+02	3.8953E+00	3.7384E-01	46
6.5000E+02	4.5000E+02	1.9320E+00	6.3270E-01	46
6.5000E+02	4.7500E+02	1.5624E+00	7.4166E-01	46
6.5000E+02	5.0000E+02	2.3617E+00	1.8311E-01	46
6.5000E+02	5.2500E+02	2.2035E+00	3.0193E-01	46
6.5000E+02	5.5000E+02	2.3348E+00	7.1015E-01	46
6.5000E+02	5.7500E+02	1.9726E+00	1.1864E+00	46
6.5000E+02	6.0000E+02	1.8016E+00	1.5128E+00	46
6.5000E+02	6.2500E+02	1.8783E+00	1.7542E+00	46
6.5000E+02	6.5000E+02	1.8888E+00	1.9112E+00	46
6.5000E+02	6.7500E+02	1.9373E+00	1.9835E+00	46
6.5000E+02	7.0000E+02	1.9399E+00	1.9915E+00	46
6.5000E+02	7.2500E+02	1.9399E+00	1.9915E+00	46
6.5000E+02	7.5000E+02	1.9419E+00	1.9904E+00	46
6.5000E+02	7.7500E+02	1.9205E+00	1.9909E+00	46
6.7500E+02	3.0000E+02	2.1579E+00	1.9559E+00	46
6.7500E+02	3.2500E+02	2.2895E+00	1.7335E+00	46
6.7500E+02	3.5000E+02	3.1590E+00	1.2584E+00	46
6.7500E+02	3.7500E+02	4.1302E+00	5.0897E-01	46
6.7500E+02	4.0000E+02	4.8955E+00	1.9873E-01	46
6.7500E+02	4.2500E+02	3.9946E+00	1.7162E-01	46
6.7500E+02	4.5000E+02	1.6875E+00	2.9608E-01	46
6.7500E+02	4.7500E+02	1.9771E+00	6.9928E-01	46

6.7500E+02	5.0000E+02	2.7792E+00	2.9569E-01	46
6.7500E+02	5.2500E+02	2.6631E+00	4.2389E-01	46
6.7500E+02	5.5000E+02	2.3361E+00	4.7006E-01	46
6.7500E+02	5.7500E+02	1.7628E+00	1.0908E+00	46
6.7500E+02	6.0000E+02	1.4387E+00	1.3902E+00	46
6.7500E+02	6.2500E+02	1.5081E+00	1.6535E+00	46
6.7500E+02	6.5000E+02	1.7118E+00	1.8654E+00	46
6.7500E+02	6.7500E+02	1.8543E+00	1.9704E+00	46
6.7500E+02	7.0000E+02	1.9205E+00	1.9909E+00	46
6.7500E+02	7.2500E+02	1.9205E+00	1.9909E+00	46
6.7500E+02	7.5000E+02	1.9205E+00	1.9909E+00	46
6.7500E+02	7.7500E+02	1.9205E+00	1.9909E+00	46
7.0000E+02	3.0000E+02	2.1527E+00	1.9015E+00	46
7.0000E+02	3.2500E+02	2.3338E+00	1.6350E+00	46
7.0000E+02	3.5000E+02	2.9014E+00	1.1628E+00	46
7.0000E+02	3.7500E+02	3.4855E+00	4.7702E-01	46
7.0000E+02	4.0000E+02	3.8367E+00	2.4739E-01	46
7.0000E+02	4.2500E+02	3.5953E+00	2.0690E-01	46
7.0000E+02	4.5000E+02	2.7185E+00	6.6320E-01	46
7.0000E+02	4.7500E+02	2.5386E+00	9.2177E-01	46
7.0000E+02	5.0000E+02	2.8569E+00	7.5390E-01	46
7.0000E+02	5.2500E+02	2.7576E+00	8.0493E-01	46
7.0000E+02	5.5000E+02	2.3031E+00	8.3646E-01	46
7.0000E+02	5.7500E+02	1.4649E+00	8.8304E-01	46
7.0000E+02	6.0000E+02	1.0067E+00	7.8577E-01	46
7.0000E+02	6.2500E+02	1.2296E+00	1.2789E+00	46
7.0000E+02	6.5000E+02	1.5536E+00	1.7311E+00	46
7.0000E+02	6.7500E+02	1.7924E+00	1.9407E+00	46
7.0000E+02	7.0000E+02	1.9137E+00	1.9893E+00	46

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7.0000E+02	7.2500E+02	1.9205E+00	1.9909E+00	46
7.0000E+02	7.5000E+02	1.9205E+00	1.9909E+00	46
7.0000E+02	7.7500E+02	1.9205E+00	1.9909E+00	46
7.2500E+02	3.0000E+02	2.1366E+00	1.7766E+00	46
7.2500E+02	3.2500E+02	2.2978E+00	1.4151E+00	46
7.2500E+02	3.5000E+02	2.5617E+00	9.9951E-01	46
7.2500E+02	3.7500E+02	2.8767E+00	5.9824E-01	46
7.2500E+02	4.0000E+02	3.3594E+00	2.3864E-01	46
7.2500E+02	4.2500E+02	3.0693E+00	2.4408E-01	46
7.2500E+02	4.5000E+02	2.6022E+00	8.2538E-01	46
7.2500E+02	4.7500E+02	2.5605E+00	1.2630E+00	46
7.2500E+02	5.0000E+02	2.6941E+00	1.4021E+00	46
7.2500E+02	5.2500E+02	2.5506E+00	1.4225E+00	46
7.2500E+02	5.5000E+02	2.0106E+00	1.3221E+00	46
7.2500E+02	5.7500E+02	1.2208E+00	9.1854E-01	46
7.2500E+02	6.0000E+02	8.2478E-01	3.3384E-01	46

7.2500E+02	6.2500E+02	1.2279E+00	1.1224E+00	46
7.2500E+02	6.5000E+02	1.6113E+00	1.6935E+00	46
7.2500E+02	6.7500E+02	1.7775E+00	1.9320E+00	46
7.2500E+02	7.0000E+02	1.9107E+00	1.9886E+00	46
7.2500E+02	7.2500E+02	1.9205E+00	1.9909E+00	46
7.2500E+02	7.5000E+02	1.9205E+00	1.9909E+00	46
7.2500E+02	7.7500E+02	1.9205E+00	1.9909E+00	46
7.5000E+02	3.0000E+02	2.1508E+00	1.6389E+00	46
7.5000E+02	3.2500E+02	2.1035E+00	1.0785E+00	46
7.5000E+02	3.5000E+02	2.4308E+00	3.8108E-01	46
7.5000E+02	3.7500E+02	2.3910E+00	4.8443E-01	46
7.5000E+02	4.0000E+02	2.5457E+00	2.9547E-01	46
7.5000E+02	4.2500E+02	2.7059E+00	2.2433E-01	46
7.5000E+02	4.5000E+02	2.4412E+00	1.0189E+00	46
7.5000E+02	4.7500E+02	2.3604E+00	1.5269E+00	46
7.5000E+02	5.0000E+02	2.3855E+00	1.7654E+00	46
7.5000E+02	5.2500E+02	2.2276E+00	1.8170E+00	46
7.5000E+02	5.5000E+02	1.7940E+00	1.7043E+00	46
7.5000E+02	5.7500E+02	1.3925E+00	1.4290E+00	46
7.5000E+02	6.0000E+02	1.2789E+00	1.2359E+00	46
7.5000E+02	6.2500E+02	1.4537E+00	1.4828E+00	46
7.5000E+02	6.5000E+02	1.7195E+00	1.8070E+00	46
7.5000E+02	6.7500E+02	1.9441E+00	1.9618E+00	46
7.5000E+02	7.0000E+02	1.9728E+00	1.9982E+00	46
7.5000E+02	7.2500E+02	1.9750E+00	1.9987E+00	46
7.5000E+02	7.5000E+02	1.9750E+00	1.9987E+00	46
7.5000E+02	7.7500E+02	1.9205E+00	1.9909E+00	46
7.7500E+02	3.0000E+02	2.0764E+00	1.6243E+00	46
7.7500E+02	3.2500E+02	1.9024E+00	1.0376E+00	46
7.7500E+02	3.5000E+02	1.3861E+00	2.0835E-01	46
7.7500E+02	3.7500E+02	1.8740E+00	3.6239E-01	46
7.7500E+02	4.0000E+02	1.7221E+00	5.7510E-01	46
7.7500E+02	4.2500E+02	1.9117E+00	9.5887E-01	46
7.7500E+02	4.5000E+02	2.0959E+00	1.3902E+00	46
7.7500E+02	4.7500E+02	2.1115E+00	1.7462E+00	46
7.7500E+02	5.0000E+02	2.0960E+00	1.9376E+00	46
7.7500E+02	5.2500E+02	2.0568E+00	1.9786E+00	46
7.7500E+02	5.5000E+02	1.8508E+00	1.9190E+00	46
7.7500E+02	5.7500E+02	1.7023E+00	1.8083E+00	46
7.7500E+02	6.0000E+02	1.6623E+00	1.7501E+00	46
7.7500E+02	6.2500E+02	1.7399E+00	1.8255E+00	46
7.7500E+02	6.5000E+02	1.8901E+00	1.9346E+00	46

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7.7500E+02	6.7500E+02	2.0186E+00	1.9891E+00	46
7.7500E+02	7.0000E+02	2.0530E+00	1.9977E+00	46
7.7500E+02	7.2500E+02	1.9750E+00	1.9987E+00	46

7.7500E+02	7.5000E+02	1.9750E+00	1.9987E+00	46
7.7500E+02	7.7500E+02	1.9750E+00	1.9987E+00	46
8.0000E+02	3.0000E+02	1.9833E+00	1.7342E+00	46
8.0000E+02	3.2500E+02	1.9163E+00	1.2846E+00	46
8.0000E+02	3.5000E+02	1.7718E+00	6.8033E-01	46
8.0000E+02	3.7500E+02	1.7905E+00	7.7339E-01	46
8.0000E+02	4.0000E+02	1.7799E+00	1.1901E+00	46
8.0000E+02	4.2500E+02	1.8345E+00	1.4894E+00	46
8.0000E+02	4.5000E+02	1.9312E+00	1.7351E+00	46
8.0000E+02	4.7500E+02	1.9581E+00	1.9057E+00	46
8.0000E+02	5.0000E+02	2.0349E+00	1.9903E+00	46
8.0000E+02	5.2500E+02	2.0392E+00	2.0018E+00	46
8.0000E+02	5.5000E+02	1.9990E+00	1.9912E+00	46
8.0000E+02	5.7500E+02	1.9521E+00	1.9653E+00	46
8.0000E+02	6.0000E+02	1.9287E+00	1.9547E+00	46
8.0000E+02	6.2500E+02	1.9622E+00	1.9695E+00	46
8.0000E+02	6.5000E+02	2.0228E+00	1.9900E+00	46
8.0000E+02	6.7500E+02	2.0530E+00	1.9976E+00	46
8.0000E+02	7.0000E+02	2.0530E+00	1.9977E+00	46
8.0000E+02	7.2500E+02	2.0530E+00	1.9977E+00	46
8.0000E+02	7.5000E+02	1.9750E+00	1.9987E+00	46
8.0000E+02	7.7500E+02	1.9750E+00	1.9987E+00	46
8.2500E+02	3.0000E+02	1.9498E+00	1.8737E+00	46
8.2500E+02	3.2500E+02	1.9418E+00	1.6315E+00	46
8.2500E+02	3.5000E+02	1.8733E+00	1.3880E+00	46
8.2500E+02	3.7500E+02	1.8198E+00	1.4179E+00	46
8.2500E+02	4.0000E+02	1.8091E+00	1.6387E+00	46
8.2500E+02	4.2500E+02	1.8537E+00	1.8256E+00	46
8.2500E+02	4.5000E+02	1.9405E+00	1.9405E+00	46
8.2500E+02	4.7500E+02	2.0016E+00	1.9902E+00	46
8.2500E+02	5.0000E+02	2.0381E+00	2.0016E+00	46
8.2500E+02	5.2500E+02	2.0396E+00	2.0019E+00	46
8.2500E+02	5.5000E+02	2.0397E+00	2.0020E+00	46
8.2500E+02	5.7500E+02	2.0524E+00	1.9975E+00	46
8.2500E+02	6.0000E+02	2.0443E+00	1.9910E+00	46
8.2500E+02	6.2500E+02	2.0331E+00	1.9721E+00	46
8.2500E+02	6.5000E+02	2.0249E+00	1.9573E+00	46
8.2500E+02	6.7500E+02	2.0296E+00	1.9661E+00	46
8.2500E+02	7.0000E+02	2.0433E+00	1.9868E+00	46
8.2500E+02	7.2500E+02	2.0529E+00	1.9975E+00	46
8.2500E+02	7.5000E+02	2.0530E+00	1.9977E+00	46
8.2500E+02	7.7500E+02	1.9750E+00	1.9987E+00	46
8.5000E+02	3.0000E+02	1.9959E+00	1.9729E+00	46
8.5000E+02	3.2500E+02	1.9738E+00	1.8841E+00	46
8.5000E+02	3.5000E+02	1.9789E+00	1.8031E+00	46
8.5000E+02	3.7500E+02	2.0028E+00	1.8179E+00	46

Geostatistical Analysis of TCE Concentration Data at the Linemaster Switch Site, Woodstock, CT.

Objective: To draw soil isopleths based on sampled data.

Given: TCE total concentration expressed as μg/kg of soil at various soil boring locations - $C_i = C(X_i, Y_i)$; $i = 1,2, ..., N$

Programs Used: GEOPACK (version 1.0), a geostatistical software system.

Steps involved in the analysis

1. Test of normality of the original concentration data, C_i, or of some appropriate transformation of C_i
2. Selection of Estimation Method
3. Analysis of variogram
4. Kriging-based estimation of concentration and its variance

Test of Normality

Before any estimation is performed it is useful to check whether the given sampling data is generated by a normally-distributed population. Therefore, simple statistical properties of C_i, such as mean, median, skewness, and kurtosis were examined. In addition, the Kolmogorov-Smirnov (KS) test for checking distributions was performed. The results clearly indicated that the original concentration data, C_i, did not originate from a normal population. This led to the search for an appropriate transformation of C_i which would yield a normally-distributed sample. The log-transformation is the most commonly used one for variables such as concentration and transmissivity. Therefore, the following transformation was done:

$$Z_i = \log_{10} C_i \quad ; \; i = 1,2,...,N \qquad (1)$$

Simple statistical properties of Z_i and the results of the KS test indicated that the sampled data, Z_i, is more likely to have originated from a normally-distributed population. Hence, the log-transformed concentration data was used in all the subsequent analyses. Please see the attached sheets for the results of simple statistical properties and the KS test.

Selection of Estimation Method

Soil isopleths can be drawn based on the sampled data using any one of several methods such as triangulation, inverse distance square, and kriging. The distinguishing feature of kriging from the other methods is that it presupposes a certain spatial structure of the variables to be estimated. This not only lends more flexibility to the estimation procedure, but also enables the generation of an estimate of uncertainty associated with each concentration estimate. This is a very significant information which can not be obtained from the other estimation or contouring methods. Therefore, kriging was the chosen method of analysis.

Analysis of Variogram

The most important step in kriging-based estimation is the determination of the spatial structure or spatial correlation of the population based on the sampled data. This involves calculating a discrete variogram from the data and fitting a smooth function to the discrete variogram. There are several smooth functions which can be fit to the discrete variogram. Based on the discrete variogram, the most appropriate ones were the exponential and the spherical models. Further evaluation of the results of the curve-fitting procedure indicated that the spherical model better describes the discrete variogram than the exponential model. The discrete variogram and the best-fit spherical model are shown in Figure 1. Please see the attached sheets for the results of the curve-fitting exercise.

Kriging-based estimation of concentration and its variance

The final step was to generate estimates of the log-transformed concentration, Z, and its variance, var(Z), at specified grid locations in order to prepare contour maps of Z and var(Z). Ordinary kriging module of GEOPACK was used to generate these estimates. Figures 2 and 3 present the contour (isopleths) maps of Z and var(Z). Isopleths of 1000, 2000, 3000, 4000, and 5000 µg/kg (in log scale) are presented in Figure 4.

Conclusions and Recommendations

Kriging was used to delineate soil isopleths at the Linemaster Switch site, Woodstock, CT. It is evident from Figure 3 that there exist areas of high uncertainty (with respect to concentration estimation). A few more samples in the marked areas would greatly reduce the uncertainty and lend more credibility to any decisions made regarding the definition of soil remediation areas.

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v. 1.00

SAMPLE STATISTICS FOR VARIABLE = Concn

NUMBER OF DATA VALUES	=	47
MEAN VALUE	=	9099.85100
MEDIAN VALUE	=	290.00000
STANDARD DEVIATION	=	31828.53000
VARIANCE	=	991500600.00000
SKEWNESS	=	5.43997
KURTOSIS (PEAKEDNESS)	=	33.91966
MINIMUM DATA VALUE	=	5.00000
MAXIMUM DATA VALUE	=	210000.00000

SAMPLE STATISTICS FOR VARIABLE = log(Concn)

NUMBER OF DATA VALUES	=	47
MEAN VALUE	=	2.63772
MEDIAN VALUE	=	2.46200
STANDARD DEVIATION	=	1.28469
VARIANCE	=	1.61530
SKEWNESS	=	-.03973
KURTOSIS (PEAKEDNESS)	=	1.90721
MINIMUM DATA VALUE	=	.69900
MAXIMUM DATA VALUE	=	5.32200

KOLOMOGOROV-SMIRNOV TEST FOR Concentration

X	NORM X	OBS. FREQ.	EXP. FREQ.	POS. DIFF	NEG. DIFF
5.0000	-.2857	.0213	.3875	.3663	.0000
5.0000	-.2857	.0426	.3875	.3450	.3663
5.0000	-.2857	.0638	.3875	.3237	.3450
5.0000	-.2857	.0851	.3875	.3024	.3237
5.0000	-.2857	.1064	.3875	.2812	.3024
5.0000	-.2857	.1277	.3875	.2599	.2812
5.0000	-.2857	.1489	.3875	.2386	.2599
10.0000	-.2856	.1702	.3876	.2174	.2387
10.0000	-.2856	.1915	.3876	.1961	.2174
13.0000	-.2855	.2128	.3876	.1749	.1961
44.0000	-.2845	.2340	.3880	.1540	.1752
55.0000	-.2842	.2553	.3881	.1328	.1541
62.0000	-.2840	.2766	.3882	.1116	.1329
68.0000	-.2838	.2979	.3883	.0904	.1117
69.0000	-.2837	.3191	.3883	.0692	.0904
144.0000	-.2814	.3404	.3892	.0488	.0701
145.0000	-.2813	.3617	.3892	.0275	.0488
160.0000	-.2809	.3830	.3894	.0064	.0277
187.0000	-.2800	.4043	.3897	.0145	.0067
226.0000	-.2788	.4255	.3902	.0353	.0141
244.0000	-.2782	.4468	.3904	.0564	.0351
268.0000	-.2775	.4681	.3907	.0774	.0561
271.0000	-.2774	.4894	.3907	.0986	.0773
290.0000	-.2768	.5106	.3910	.1197	.0984
346.0000	-.2750	.5319	.3916	.1403	.1190
680.0000	-.2645	.5532	.3957	.1575	.1362
1313.0000	-.2447	.5745	.4034	.1711	.1498
1825.0000	-.2286	.5957	.4096	.1861	.1649
1892.0000	-.2265	.6170	.4104	.2066	.1853
2000.0000	-.2231	.6383	.4117	.2266	.2053
2300.0000	-.2136	.6596	.4154	.2442	.2229
3000.0000	-.1916	.6809	.4240	.2568	.2356
3700.0000	-.1697	.7021	.4326	.2695	.2482
3858.0000	-.1647	.7234	.4346	.2888	.2675
4022.0000	-.1595	.7447	.4366	.3081	.2868
5400.0000	-.1162	.7660	.4537	.3122	.2910
6100.0000	-.0943	.7872	.4625	.3248	.3035
7700.0000	-.0440	.8085	.4825	.3261	.3048
8000.0000	-.0346	.8298	.4862	.3436	.3223
9100.0000	.0000	.8511	.5000	.3511	.3298

9656.0000	.0175	.8723	.5070	.3654	.3441
9800.0000	.0220	.8936	.5088	.3848	.3636
9900.0000	.0251	.9149	.5100	.4049	.3836
24000.0000	.4681	.9362	.6802	.2560	.2347
43200.0000	1.0714	.9574	.8580	.0994	.0782
57600.0000	1.5238	.9787	.9362	.0425	.0212
210000.0000	6.3120	1.0000	1.0000	.0000	.0213

Kolomogorov-Smirnov Test Statistic (ß) = .405

REJECT The Null Hypothesis At The (.10) Level
Kolomogorov-Smirnov Critical (Test) Value = .117

REJECT The Null Hypothesis At The (.05) Level
Kolomogorov-Smirnov Critical (Test) Value = .129

REJECT The Null Hypothesis At The (.01) Level
Kolomogorov-Smirnov Critical (Test) Value = .150

Null Hypothesis: Observed Distribution Same As A Normal Distribution

NOTE: results use the intrinsic hypothesis - i.e. the theoretical distribution uses the mean and std from the sample population (This is the usual case)

KOLOMOGOROV-SMIRNOV TEST FOR log(Concentration)

X	NORM X	OBS. FREQ.	EXP. FREQ.	POS. DIFF	NEG. DIFF
.6990	-1.5091	.0213	.0656	.0444	.0000
.6990	-1.5091	.0426	.0656	.0231	.0444
.6990	-1.5091	.0638	.0656	.0018	.0231
.6990	-1.5091	.0851	.0656	.0195	.0018
.6990	-1.5091	.1064	.0656	.0407	.0195
.6990	-1.5091	.1277	.0656	.0620	.0407
.6990	-1.5091	.1489	.0656	.0833	.0620
1.0000	-1.2748	.1702	.1012	.0690	.0477
1.0000	-1.2748	.1915	.1012	.0903	.0690
1.1140	-1.1861	.2128	.1178	.0950	.0737
1.6430	-.7743	.2340	.2194	.0147	.0066
1.7400	-.6988	.2553	.2423	.0130	.0083
1.7920	-.6583	.2766	.2552	.0214	.0002
1.8330	-.6264	.2979	.2655	.0323	.0111
1.8390	-.6217	.3191	.2671	.0521	.0308
2.1580	-.3734	.3404	.3544	.0140	.0353
2.1610	-.3711	.3617	.3553	.0064	.0149
2.2040	-.3376	.3830	.3678	.0152	.0061
2.2720	-.2847	.4043	.3879	.0163	.0050
2.3540	-.2209	.4255	.4126	.0129	.0083
2.3870	-.1952	.4468	.4226	.0242	.0029
2.4280	-.1632	.4681	.4352	.0329	.0116
2.4330	-.1594	.4894	.4367	.0527	.0314
2.4620	-.1368	.5106	.4456	.0650	.0438
2.5390	-.0768	.5319	.4694	.0625	.0413
2.8330	.1520	.5532	.5604	.0072	.0285
3.1180	.3738	.5745	.6457	.0713	.0925
3.2610	.4852	.5957	.6862	.0905	.1118
3.2770	.4976	.6170	.6906	.0736	.0949
3.3010	.5163	.6383	.6972	.0589	.0802
3.3620	.5638	.6596	.7135	.0540	.0752
3.4770	.6533	.6809	.7432	.0624	.0836
3.5680	.7241	.7021	.7655	.0634	.0847
3.5860	.7381	.7234	.7698	.0464	.0677
3.6040	.7522	.7447	.7740	.0293	.0506
3.7320	.8518	.7660	.8028	.0369	.0582
3.7850	.8930	.7872	.8141	.0268	.0481
3.8860	.9717	.8085	.8344	.0259	.0472
3.9030	.9849	.8298	.8377	.0079	.0292
3.9590	1.0285	.8511	.8481	.0029	.0184

3.9850	1.0487	.8723	.8528	.0195	.0018
3.9910	1.0534	.8936	.8539	.0397	.0184
3.9960	1.0573	.9149	.8548	.0601	.0388
4.3800	1.3562	.9362	.9125	.0237	.0024
4.6350	1.5547	.9574	.9400	.0175	.0038
4.7600	1.6520	.9787	.9507	.0280	.0067
5.3220	2.0894	1.0000	.9817	.0000	.0029

Kolomogorov-Smirnov Test Statistic (ß) = .112

ACCEPT The Null Hypothesis At The (.10) Level
Kolomogorov-Smirnov Critical (Test) Value = .117

ACCEPT The Null Hypothesis At The (.05) Level
Kolomogorov-Smirnov Critical (Test) Value = .129

ACCEPT The Null Hypothesis At The (.01) Level
Kolomogorov-Smirnov Critical (Test) Value = .150

Null Hypothesis: Observed Distribution Same As A Normal Distribution

NOTE: results use the intrinsic hypothesis - i.e. the theoretical distribution uses the mean and std from the sample population (This is the usual case)

```
******************************************************************************
*                                                                            *
*      SAMPLE SPATIAL-CORRELATION CALCULATION -   SEMIVARIOGRAM              *
*                                                                            *
* * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                                            *
*  Maximum total concentrations of TCE expressed in terms of soil concent   *
*  Linemaster Switch site                                                    *
*  May 08, 1996                                                              *
*                                                                            *
*    DATE: 05/22/1996                              TIME: 14:41 pm            *
******************************************************************************

+----------------------+               +------------------------+
| ANGLE =    .00 deg.  |               | DANGLE = ± 90.00 deg.  |
+----------------------+               +------------------------+
```

LAG	No. OF COUPLES	DISTANCE	SEMIVARIOGRAM (lConcn)	DRIFT (lConcn)
1	14	8.09	.8626E+00	-.1249E+00
2	73	22.73	.3831E+00	.2300E-01
3	103	41.07	.8607E+00	.3000E-02
4	94	63.83	.1304E+01	-.4001E+00
5	85	81.27	.1872E+01	-.7666E+00
6	96	103.24	.2094E+01	-.1021E+01
7	88	124.48	.2167E+01	-.1027E+01
8	77	144.10	.2224E+01	-.5606E+00
9	54	166.06	.1745E+01	-.7589E+00
10	62	186.31	.2042E+01	.4261E+00
11	74	206.30	.1537E+01	.3226E+00
12	53	228.30	.1268E+01	.3729E+00
13	42	248.11	.1649E+01	.4824E+00
14	39	268.53	.1730E+01	-.4555E+00
15	27	290.56	.2102E+01	.1589E-01

END OF PROBLEM

1

```
1*******************************************************************************
*                                                                  *
*                NON-LINEAR LEAST SQUARES ANALYSIS                 *
*                                                                  *
*          Fit A Variogram Model To The Sample Variogram           *
*            Using A Non-Linear Least-Squares Analysis             *
*                                                                  *
*******************************************************************************
```

INPUT PARAMETERS
===============

MODEL NUMBER.................................... -1
NUMBER OF COEFFICIENTS........................... 3
MAXIMUM NUMBER OF ITERATIONS..................... 20
RATIO OF COEFFICIENTS CRITERION.................. .0005

OBSERVED DATA

=============

OBS. No.	No. COUPLES	DISTANCE	GAMMA
1	73	.2273E+02	.3831E+00
2	103	.4107E+02	.8607E+00
3	94	.6383E+02	.1304E+01
4	85	.8127E+02	.1872E+01
5	96	.1032E+03	.2094E+01
6	88	.1245E+03	.2167E+01
7	77	.1441E+03	.2224E+01
8	54	.1661E+03	.1745E+01
9	62	.1863E+03	.2042E+01
10	74	.2063E+03	.1537E+01
11	53	.2283E+03	.1268E+01
12	42	.2481E+03	.1649E+01
13	39	.2685E+03	.1730E+01
14	27	.2906E+03	.2102E+01

ITER NO	NUGGET	SILL-NUGGET	RANGE	SSQ	MODEL
0	.4449E+00	.2002E+01	.1453E+03	2.614E+00	EXPONENT
1	.3143E+00	.1509E+01	.4356E+02	1.657E+00	EXPONENT
2	.2706E+00	.1590E+01	.4892E+02	1.599E+00	EXPONENT

3	.2074E+00	.1651E+01	.4601E+02	1.559E+00	EXPONENT
4	.9296E-01	.1761E+01	.4292E+02	1.496E+00	EXPONENT
5	.4754E-01	.1816E+01	.4429E+02	1.466E+00	EXPONENT
6	.2219E-01	.1840E+01	.4362E+02	1.454E+00	EXPONENT
7	.3105E-03	.1861E+01	.4306E+02	1.443E+00	EXPONENT
8	.0000E+00	.1861E+01	.4306E+02	1.443E+00	EXPONENT

CORRELATION MATRIX
===============

	1	2
1	1.0000	
2	.6388	1.0000

NONLINEAR LEAST-SQUARES ANALYSIS: FINAL RESULTS
==

				95% CONFIDENCE LIMITS	
VARIABLE	VALUE	S.E.COEFF.	T-VALUE	LOWER	UPPER
RANGE	.43058E+02	.14401E+02	.2990E+01	.1168E+02	.7444E+02
SILL-N	.18614E+01	.13345E+00	.1395E+02	.1571E+01	.2152E+01
NUGGET	.00000E+00	--	--	--	--

APPENDIX E

APPENDIX E

MEETING SUMMARY MAY 16, 1996

MEETING SUMMARY

Conceptual Design Meeting
May 16, 1996

The following will summarize the discussions held during the Conceptual Design meeting on May 16, 1996. The following notes will serve as the preliminary Conceptual Design Report, and form the basis of the Conceptual Design. A summary of the items on which there was concurrence is attached. Also attached is a copy of the map showing utility locations east of the manufacturing facility that was sent via facsimile to the participants on May 17. Participating in the meeting was the following:

Elise Jackabhazy, EPA
Mary Jane O'Donnell, EPA
Dom DiGiulo, EPA
V. Ravi, Dynamac
Mike Beskind, DEP
Gary Kennett, Linemaster
Mac White, Linemaster
Cinthia McLane, M&E
Heather Vick, M&E
Warren Diesl, M&E
Jeff Ford, M&E
Larry Murdoch, FRx
Mike Marley, Envirogen
Chris Klemmer, F&O
Tim Whiting, F&O
Fred Mueller, F&O
Dave Bramley, F&O

I. General Considerations

Dave Bramley made an opening presentation in which he stated the purpose of the Conceptual Design was to develop an approach to remediate the soil and maintain groundwater migration control. The scope would be the Phase I area, which would be discussed later in the meeting. The objective is to maximize containment of mass removal. Dave then explained that the Conceptual Design developed by F&O uses the following criteria:

Well spacing	40 feet OC
Horizontal fracture separation	5 to10 feet
Vertical fracture spacing	7.5 feet
Fracture radius	15 feet
Offset from building	10 feet

All installed wells will be fractured and mini-injection wells will address SVE dead zones. Dave noted that it was important to obtain concurrence on a Conceptual Design if the goal of achieving construction during the Linemaster shutdown was to be achieved. He noted that the timeframe was very short and it would be necessary to retain contractors soon to perform the interior construction. Dave also briefly discussed monitoring requirements and indicated that knowing when remediation was complete would be an important consideration.

Fred Mueller then followed with an elaboration on the Conceptual Design. Draft copies of Fred's presentation were distributed to the participants. Fred proposed a Phase I target area that would include the interpolated 4,000 ug/kg TCE isopleth. The Conceptual Design presented includes three interior wells and five exterior wells. Also, the existing fracture FW-A would be included in the final design. Mike Beskind asked about the possibility of individually accessing the fractures. Fred explained that a 6-inch diameter casing would not be large enough to accommodate the number of pipes that would be necessary to access the individual fractures. He noted that in a five fracture well, the top and bottom fractures likely would be accessed together, and the remaining fractures accessed by the remaining pipe.

Tim then presented an analysis of concurrent versus sequential well installation and indicated that the time required to do the sequential construction would be more than 50 percent longer than the concurrent approach. Mike Beskind questioned the assumptions used in Tim's scheduling. He asked if the subsequent well could be drilled after the fracture radius of the previous well was determined via uplift. He indicated that all the piezometers could be constructed following fracturing, which would minimize the number of mobilizations. Tim responded that it would be more appropriate to have all of the data, which would include the uplift information and information gained by locating the fractures via borings. There followed a short discussion about locating fractures. Tim mentioned that one option that was being evaluated was sonic drilling. Mike Beskind asked for some information on it. Fuss & O'Neill will provide copies of some of the literature.

After lunch, Elise stated that there was still much EPA disagreement about the limits of Phase I and which isopleth should indicate the limit for remediation. Elise said it is difficult to agree with the F&O proposed 4,000 ug/kg contour because the EPA data krige under the building shows uncertainty in the northwest corner and south of the dry well. The location of the exterior wells proposed by F&O however, generally include the area of concern except to the south and northwest. Elise indicated that there were several data gaps and it would be beneficial to include another boring in the northwest corner of the building. Additional remediation under the building; however, would occur if there were a Phase II. There was concurrence on the installation of three interior wells and that the goal should be to install the wells during the July shutdown. The discussion then was focused on the agenda prepared by the EPA for the meeting.

II. Phase 1 Delineation

Dom discussed the data krige that was performed on the soil analyses from the boring program. He indicated that the data were filtered and some points were removed. He noted that the deeper contamination away from the dry well was the result of groundwater flow. The soil information from borings less than 15 feet was eliminated and a minimum of four samples per boring to depth was required for the data to be included. He noted that a partitioning coefficient of 0.09 times the groundwater concentration was used to estimate the soil concentration. The kriging analyses also provided variance distribution indicating areas of uncertainty. Dr. Ravi then explained the kriging analysis noting that log transformed data was used, and the data were kriged using Geopack which estimates the concentration and variances of the concentration.

Based on 47 data points, the Phase I area is generally oval in nature, oriented northwest/southeast around the EPA well cluster and extending toward the location of FW-B. A copy of the map is attached. He indicated that the data may suggest two sources; the dry well area and an area in the vicinity of the EPA dry well cluster. To address some of the data gaps, samples will be collected during the installation of the interior wells at 5-foot intervals and the data can be reevaluated. In addition, Dom indicated that a "what-if" kriging could be to simulate hypothetical concentrations within the building. Elise noted that additional future remedial design efforts would focus on the dead zones between wells. She noted that there was no consensus on the mini-well locations and consideration would be deferred to later in the summer.

Regarding the location of the exterior wells; again, data would be modified to test hypothetical concentrations. There also is no agreement yet on the need to include groundwater extraction from the shallow bedrock aquifer. M&E will review the issue with Tim and make recommendation.

III. Under the Linemaster Facility

It was agreed that the first phase of the work would include three wells within the building. All would be fractured, and there would be no conventional DVE wells. It will be difficult to monitor the performance of the fractures because the building slab likely will mask the uplift data. Tilt meters likely will be used to minimize any damage to the floor slab.

IV. Fracture Techniques/Fracture Delineation

Mike Marley suggested that a relationship be developed between the conditions outside of the building and those inside. The effectiveness of mass removal outside the building may be able to be related to the interior. Regarding methods to better locate the fractures, Tim indicated that he would pursue the availability of a third type of sand. Regarding modeling, Larry said that he had been discussing inverse fracture estimation with a California firm. This firm could review the uplift data developed and develop a 3-D image of the fracture (if the modeling functioned as anticipated). Larry agreed that modeling would not produce a unique conclusion. There could be several potential solutions and there would have to be a general agreement about the one selected. Also, there would have to be constraints placed on the interpretation of the data to limit options. Larry said he would continue to investigate the feasibility of using this type of modeling and evaluate the cost versus potential return.

Larry discussed that the best way to influence fractures is to influence the azimuth with a downhole device that he has used at another site. He acknowledged that there is little opportunity to limit the dip. Heather asked if it could be used at Linemaster before the shutdown. Fred noted that it wasn't necessary to try the techniques before the shutdown. Only the wells have to be drilled during the shutdown. Larry did note that the packer had to be modified. Tim noted that trying it at another site with different soil conditions might not confirm the usefulness at Linemaster because conditions at a remote site may differ significantly. The location of a "test well" to try the fracture orientation modifications was discussed. Based on the exterior wells proposed by F&O, the southernmost fracture well would be the one selected as the test well. Larry noted that the packer would have to be modified to implement the control procedures. Likely, the controls would consist of three separate injections at the same elevation The areas not receiving the injection would be baffled from the active injection point. Larry suggested that the exterior work be done first to learn the fracturing style before moving inside. It is likely that the building will only be able to accommodate two fractures below the floor; one near the bedrock interface, and one to be determined based on the contaminate profile during the drilling, probably 20 to 25 feet deep.

VI. Dewatering/Monitoring Progress

Tim noted the fracture DVE wells spaced at 70 feet on-center will be sufficient to dewater the site based on our modeling. Inclusion of wells constructed in the shallow bedrock would not significantly improve the ability to dewater. He noted that deep bedrock pumping has increased the downward flow potential. The fracture wells will be extended into the shallow bedrock to allow fracturing as close to the bedrock surface as possible and, consequently, assist in dewatering. Warren noted that inclusion of shallow bedrock wells would impact the shallow bedrock aquifer. He noted that the area not dewatered would be included in the category of "dead zones." Tim responded that the shallow bedrock wells would not negatively impact dewatering but, likely, would not help. He suggested that if a shallow bedrock well would be installed that only one would be required. Dom asked about pumping from MW-10sb. Chris noted that well could be included in the extraction network with little difficulty; though he did note that there would be a relatively low flow.

The discussion moved to methods to assess soil moisture content. There was considerable discussion about the neutron probe and how the wells would have to be constructed. There was question about how many access wells would be required to monitor the soil moisture content. Dom noted the cost of a unit is approximately $4,300 plus one day of training, and an NRC license is required. Gary was concerned about the cost and, with the additions to the scope, Elise noted that the alternative would be drilling. Mike added that the probe could be used to monitor dewatering. If dewatering can't be achieved, then air cannot move through the soil. Alternatives were discussed, but it generally was agreed that the neutron probe was the most appropriate for the site-specific conditions. Mike M. noted that it was the only currently proven technique. He suggested that it be evaluated on known soil samples. It was decided to continue the discussion at a later date.

To address potential free-phase liquid, Tim stated that the portion of the casing extended into bedrock in the fracture wells could be constructed of stainless steel. He did note that PVC piping would be suitable for removal of the groundwater because it would not be in constant contact with free phase TCE. The treatment system would also include a phase separator to remove any free phase liquid.

Regarding dead zones, Tim noted that there would be no dewatering the dead zones below the lowest fracture. Extending the fracture well casing into the bedrock would allow the lowest fracture to be deeper. No agreement was reached and the matter was tabled for further discussion later.

VII. Effectiveness of Hydraulic Fracture/Air Flow Distribution

To detect short circuiting and assess mass transfer, a tracer gas study could be used. The slope of the breakthrough curve can provide information on advective vs. diffusive flow. Dom offered to conduct such a study. Mike Beskind asked if it would be useful at Linemaster due to the complexities of the soil. Dom noted it would be a research type project and could be done at little charge to Linemaster. Mike Marley suggested that first the gross data should be evaluated using air flow and VOC concentration as a tracer. If additional information was required, then, a separate tracer gas could be used. Larry suggested that the tracer test be limited to the highest priority flow paths. Mike Marley suggested that it be used to answer questions or support hypotheses.

Mike B. noted that the degree of mass removal was uncertain, and asked if the wells could be configured to accept heat/steam injection. Fred responded that PVC casing was not amenable to heat. Larry added that the mini-wells could be used to inject warm air or steam in the future. The method of constructing these types of systems requires the use of PVC pipe inside the well. Steam or hot air injection would be accomplished via the air injection wells which would be constructed using steel pipe. Mike Marley added that the impacts of the hot air could be modeled once the system is operational and performance data are available. Hypothetical situations that could be evaluated with a model could indicate whether injection were feasible and if the remedial duration would be shortened. Larry noted that FRx would be doing a similar project at another site in Ohio.

VIII. Fracture Well Specifics

Larry indicated that the mixture might be something like 10 gallons of liquid per 100 pounds of sand and use 1,000 to 2,000 pounds of sand per fracture. He indicated, however, that it would be a learn-as-you-go process; the mixture may be modified from fracture to fracture. Larry also discussed the merits of the packers and indicated that it was easier to use the packers than to use individual wells for each fracture interval. There are methods to check for leaks in the seals, but it wouldn't be necessary unless there is a problem in the field. He said if the seal leaks the packer can be removed and the seal fixed.

IX. Closure Compliance Monitoring

Dom discussed some of the aspects of his paper. Dom noted that the biggest uncertainty in the remedial activity in the shallow bedrock aquifer remediation. If the concentration of VOCs in the shallow bedrock aquifer stays high while the soils are cleaned via DVE then the soils will be recontaminated by the diffusion from the groundwater back into the soil. He suggested that the remediation should be kept in balance and the goal should be to halt the mass flux to the shallow bedrock groundwater. He noted that the concentration of TCE in the till should be equal to the concentration of TCE in the shallow bedrock when remediation is complete. Mike Marley noted that a caveat was that it would be necessary to prove that SVE is a feasible technology. Elise indicated that more thought should be given to the inclusion of shallow bedrock wells, unless pumping them could create negative offsite impacts. It will be necessary to develop a consensus on performance monitoring and closure monitoring.

Dom suggested a new topic; he offered to perform some on-site testing to support a paper that he was preparing on finite well radius and low permeability media. He will need to evaluate the pneumatic permeability in the deep till after the site has been dewatered. He noted that there would be little cost to Linemaster.

In summary, there was general agreement on the location of the interior fracture wells and preliminary agreement on the exterior fracture wells. Elise requested a plan showing potential interferences with existing utilities. Dave said he would prepare such a sketch and transmit it early in the week. The goal is to install the interior wells and construct the necessary piping during the July shutdown.

A conference call has been scheduled for May 23 at 9:00 a.m. to continue discussions on the remaining items. Elise will prepare the agenda. The following, however, are items on which concurrence was not achieved or were not discussed.

1. Phase I extent/coverage area
2. Extending extraction wells into bedrock
3. Dewatering dead zones
4. Shallow bedrock aquifer pumping
5. Control of overburden aquifer
6. Method of fracture location
7. Monitoring
8. Mini-injection wells

Attachments

1. Map showing well locations, isopleths, and utilities
2. Map showing proposed monitoring points
3. Map of EPA data krige
4. Map of data krige uncertainty
5. Draft list of items in agreement

APPENDIX F

APPENDIX F

PHASE 1 REMEDIATION ACTIVITIES - CONCEPTUAL DESIGN MEETING MAY 29, 1996 (EPA)

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
REGION I
J.F. KENNEDY FEDERAL BUILDING, BOSTON, MASSACHUSETTS 02203-2211

RECEIVED
JUN 3

May 29, 1996

Mr. Gary Kennett
Project Coordinator
Linemaster Switch Corporation
29 Plaine Hill Road / P.O. Box 238
Woodstock, CT 06281

Re: Linemaster Switch Corporation Superfund Site: Phase I Remediation Activities - Conceptual Design Meeting

Dear Mr. Kennett:

The U.S. Environmental Protection Agency (EPA) and the Connecticut Department of Environmental Protection (CT DEP) have reviewed the May 16, 1996 Conceptual Design Phase I Remediation Activities proposal presented by Fuss & O'Neill, Inc. on behalf of the Linemaster Switch Corporation. Our review was conducted in accordance with the Consent Decree issued in United States of America and The State of Connecticut v. Linemaster Switch Corporation, Inc., D. Conn. 1995, Civil Action Nos. 3:94CV01709, 3:94CV01710, (the "Consent Decree") for Remedial Design and Remedial Action, entered on January 4, 1995.

The presentation was well coordinated, and very informative. During the meeting, EPA and CT DEP agreed to Fuss & O'Neill's approach to a "grid plot" for installing the fracture wells. While both agencies agree that there would be a potential benefit for a sequential drilling program; the expense for undertaking such a project far outweighs the benefit. EPA acknowledges that Linemaster is undertaking a series of innovative technologies, and respectfully understands the uncertainties and limitations of the hydraulic fracturing to be performed at the site.

While all parties at the technical meeting were in agreement with the well installation methodology and spacing, we were not all in agreement regarding either the methodology for defining the Phase I area or how to best design/construct/remediate the "dead zones" in between and beneath the fractures. Neither the EPA nor the CT DEP agree with the remediation boundaries delineated by Fuss & O'Neill during their presentation. Both agencies disagree with the rationale for choosing a 4,000 μg/kg isopleth for the boundary of the Phase I area.

EPA and CT DEP have agreed to a 1,000 μg/kg isopleth using the geostatistical method of data kriging to be used to define the boundaries. This 1,000 μg/kg limit for defining the Phase I is based upon the same rationale defined in our April 12, 1996 letter to you (re: Linemaster Switch Corporation Superfund Site ("Linemaster"): Feasibility of DVE and Soil Fracturing at the Linemaster Switch Site).

The purpose of this phase of the remediation is to remove as much of the contaminant mass from the Phase I soils as is technically feasible (please see the enclosed figure which delineates the Phase I boundary). The 1,000 μg/kg TCE boundary in soils is already *200 times* greater than the 5 μg/kg level specified in the Record of Decision (ROD). EPA, therefore, rejects Linemaster's proposed 4,000 μg/kg boundary, as we believe that it is technically feasible to achieve a significant amount of TCE removal within the area defined by the 1,000 μg/kg isopleth provided as an attachment to this letter.

Linemaster must, therefore, submit a new proposal for a well installation "grid" layout based upon the Phase I delineation provided in the enclosed figure. With respect to the data gaps under the building, EPA and CT DEP can only assume that the two isopleths (labeled 'A' and 'B' on the enclosed diagram) are connected by an somewhat elliptical shape through the data gaps under the Linemaster facility (labeled 'C' on the enclosed diagram. Phase I, therefore, is defined as the union of the areas labeled A, B and C). Should data become available through sampling, however, EPA and CT DEP might then conclude that these areas are either interconnected or are two discrete areas of contamination. As a result, EPA and CT DEP require the following:

- During the July shutdown, Linemaster must install three wells (and take soil samples) inside the building (taking into consideration the 1,000 μg/kg isopleth provided as an attachment to this letter). Again, Linemaster shall be responsible for choosing the locations and submitting the "grid" well layout to EPA and CT DEP for review/comment and approval.

- During the July shutdown, Linemaster must also concurrently drill bore holes to obtain soil samples from under the Linemaster facility using a separate drill rig from three areas to be defined by Linemaster with review and approval by EPA and CT DEP. The goal of these three samples is to determine the extent of contamination under the Linemaster facility and to better define the Phase I remediation. If three borings cannot be drilled during the shutdown (due *only* to time and/or drilling constraints), EPA and CT DEP shall review the data obtained during this 1996 July shutdown and evaluate if additional samples need to be obtained during the 1997 July shutdown at the Linemaster facility. Additionally, depending upon the borehole locations selected by Linemaster, these borehole locations must be used to install either wells or vapor probes.

- Linemaster may submit a written request that the area that falls within the 1,000 μg/kg isopleth to the northwest of the building (defined by the 1,000 μg/kg isopleth labeled 'B' on the enclosed diagram) be addressed during the 1997 - 1999 construction season(s) following a review and assessment of the performance of the technologies being implemented in the "former drywell area" and under the facility (defined by the 1,000 μg/kg isopleth labeled 'A' on the enclosed diagram). Based upon the existing data gaps, however, the area of the entire 1,000 μg/kg isopleth (which is represented by the union of areas 'A', 'B' and 'C' on the enclosed diagram) shall be considered as the Phase I Remediation boundary. It is also acceptable to EPA and CT DEP for Linemaster to proceed with remediating the entire Phase I area, should Linemaster so choose.

Following the installation of the fracture wells in the 1,000 $\mu g/kg$ area, the "dead zones" must be defined and evaluated for additional remediation. As for remediation of the "dead zones" in between fractures and in the deep till below the lowest fractures, EPA and CT DEP do not agree with the proposal to use air injection. Though EPA and CT DEP understand Linemaster's logic for using injection/air sparging methods, they shall not be considered as a remedial option because injection/air sparging may move and/or push the air and water to locations that we cannot map, locate or ensure will be collected by the remedial system.

The 50% and 100% Remedial Design shall, therefore, focus on the remediation of the "dead zones" within the Phase I area. The remediation solutions should be designed to enhance the Phase I remediation, without interfering with the hydraulically fractured wells that shall be installed during summer 1996. These "dead zones" may need to be remediated with a series of conventional Dual Vacuum Extraction (DVE) wells which may or may not need to be extended into the shallow bedrock to help dewater the Phase I area. These wells may or may not need to be hydraulically fractured. These wells may or may not need to be steam injected. Again, this portion of the remediation is for Linemaster to design, and for EPA and CT DEP to review and ultimately approve.

Additionally, once Linemaster submits a Phase I "grid" well-plot to EPA and CT DEP for review and approval, EPA and CT DEP shall then provide Linemaster with locations and requirements for drilling and installing a series of monitoring/compliance points for evaluation of the Phase I remedial system. In an attempt to provide additional cost savings to Linemaster, EPA believes that it is best to define these borehole locations prior to FRx drilling to locate their hydraulic fractures. In many instances, a monitoring/compliance point and a borehole to locate fractures may be drilled as the same borehole.

Again, it must be noted that the on-site air stripper must be retrofitted with off-gas emission controls as defined in the ROD *prior* to commencing the operation of the DVE wells. The air and water streams from the Phase I area shall, most likely, be highly contaminated during the start-up activities, and the air stripper shall not be permitted to release TCE (an ozone pre-cursor) to the atmosphere. Please refer to the ROD if you have additional questions.

Please do not hesitate to call me at (617) 573-5760 regarding any of these comments.

Sincerely,

Elise I. Jakabházy,
Project Manager

encl.
cc: Mary Jane O'Donnell, US EPA
Martin Beskind, CT DEP
Cinthia McLane, M&E
David Bramley, F&O

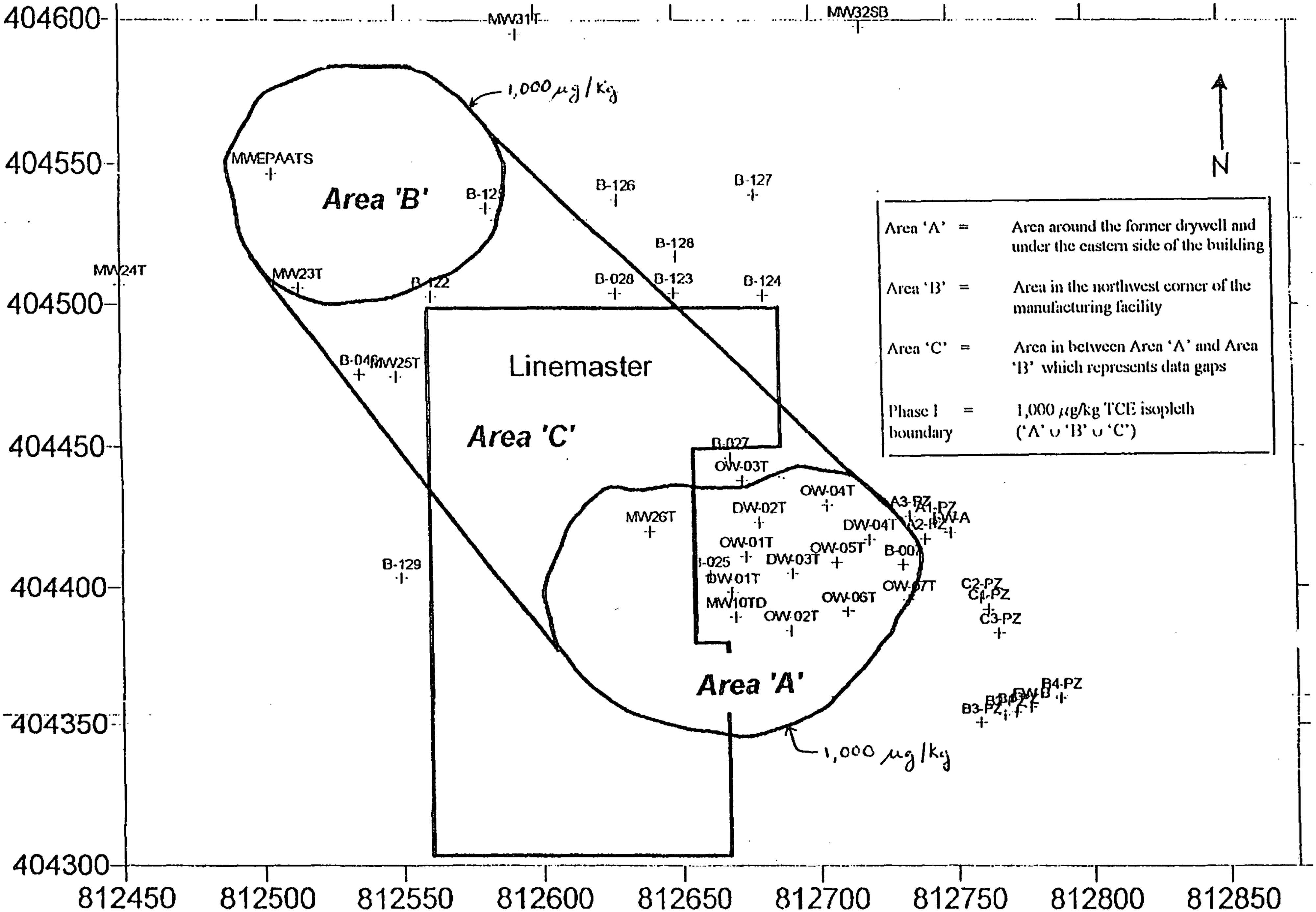


Figure 1: 1,000 µg/kg TCE Isopleth (based upon geostatistical data kriging)

APPENDIX G

APPENDIX G

AIR FLOW RATE DETERMINATION LETTERS

Fuss & O'Neill Inc. *Consulting Engineers*

146 Hartford Road, Manchester, CT 06040-5921
TEL 860 646-2469 FAX 860 643-6313

1200 Converse Street, Longmeadow, MA 01106-1721
TEL 413 567-9886 FAX 413 567-8936

Providence, RI TEL 401 828-3510

May 30, 1997

Ms. Elise Jakabházy, RPM
U.S. Environmental Protection Agency, Region 1
Office of Site Remediation & Restoration
ME/VT/CT Superfund Section (HBT)
JFK Federal Building
Boston, MA 02203-2211

RE: Linemaster Switch Corp.
Fracture Wells Air Flow Rates

Dear Ms. Jakabházy:

The following will respond to the letter dated April 24, 1997, from Martin Beskind, regarding vacuum air flow rates from the fractured wells.

1. *An allowance of 10% has been included to account for flows from lower fractures based on the 1995 vacuum and flow test data. However, only partial dewatering had been achieved during the pilot test - mainly in the upper till. Air flow rates from the deep till under such conditions were necessarily much lower than flowrates that would be reached after long-term dewatering. Please consider revision of your estimate to include flows expected for fully dewatered, partially dried, deep till.*

The air flow recovery rate from a fractured well has been computer simulated using an air permeability of 2.89×10^{-9} cm^2 for the entire till unit. Below a depth of 12 feet however, the permeability is believed to be closer to 1.5×10^{-9} cm^2 . Therefore, the flow rates estimated for fractures present in the deeper till unit are conservatively high. Air flow profiles in the shallow till indicate that planar flow is dominant, whereas cylindrical flow with major cylindrical pressure gradients at the outer edge of the fractures is characteristic of the deeper till deposits. In addition, there were appreciable radial pressure drops within the fractures that limited the vacuum force for flow.

Simulations estimate the flow rate from the top fracture to be approximately 12 scfm. A flow rate of 10 scfm was estimated for each of the lower fractures. Upon the successful fracturing of the five FW-F wells, a total of 24 hydraulic fractures will have been created from the seven Phase 1A area recovery wells. The number of fractures in each recovery well ranges from two (FW-H and FW-J) to five (FW-Fs) with an average of 3.4 hydraulic fractures per recovery well. Therefore, the average air flow recovery rate from each fractured well was estimated to be 36 scfm. We believe that the 36 scfm estimate is conservatively high (due to higher permeability used for the lower till). As an additional conservative measure, fractured recovery well VOC loading rates were calculated assuming a flow rate of 40 scfm. This rate also was used to size the vacuum blower and other components of the SVE system.

2. Please consider including additional air flow to account for short-circuiting of atmospheric air via the surface vents and fracture interconnections formed during the fracturing. Significant additional air flow might have to be drawn via these short circuits in order to provide adequate air flow through the till matrix.

We agree that surface vents potentially may serve as inlets for ambient air and could result in higher than anticipated air flow recovery rates. The problems of ambient air short-circuiting due to surface vents have been or will be (boring B-25) addressed by backfilling with impermeable materials the boreholes where fracture fluid was observed to vent during hydraulic fracturing. Former monitoring points in the Phase IA area previously have been abandoned in similar fashion. Fracture fluid vents observed adjacent to the fractured recovery well casings will be addressed by the placement of impermeable seals in the base of the well vaults that will be installed at each recovery well location.

With the elimination of the identified surface vents, we do not expect that fracture interconnection, regardless of degree, will to lead to an increase in air flow recovery rates. As indicated in our response to Comment 1, for design purposes we have included an approximate 10 percent contingency to our already conservatively estimated air recovery rate to allow for unanticipated increased air recovery rates.

Additionally, please note that unlike the 1995 Pilot Test, the design of the fractured recovery well internals proposed for the Phase 1A area wells allows the use of up to four pipes to access the hydraulic fractures. As no more than four fractures were created in each of the recovery wells fractured in November 1996, the design potentially allows for independent access to each fracture. We believe that this design flexibility will enable effective operation of the DVE even if fracture interconnection is found to be significant. As indicated in the May 14, 1997 Fracture Testing Work Plan, fracture testing results and confirmation boring results will be evaluated thoroughly prior to the final design and installation of the internal plumbing in each fractured recovery well.

If you have questions, please do not hesitate to call.

Sincerely,

David L Bramley

David L. Bramley, P.E.
Project Manager

c. Gary Kennett - Linemaster
Martin Beskind - DEP
Cinthia McLane - M&E
Mike Marley

86088\A7\DLB0527A.WPD
Corres.

Fuss & O'Neill Inc. *Consulting Engineers*

146 Hartford Road, Manchester, CT 06040-5921
TEL 860 646-2469 FAX 860 643-6313

1200 Converse Street, Longmeadow, MA 01106-1721
TEL 413 567-9886 FAX 413 567-8936

Providence, RI TEL 401 828-3510

June 12, 1997

Mr. Martin Beskind
Environmental Analyst
DEP-PERD
79 Elm Street
Hartford, CT 06106-5127

RE: Linemaster Switch Corp.
SVE Air Flow Rates/VOC Loads

Dear Mr. Beskind:

The following will respond to the issues raised in your e-mails of June 2, 1997. These were generated in response to our letters of May 30, which addressed the subject design criteria. We will address the air flow rate issue first because it relates directly to the VOC loading question. Before addressing the specific questions in the E-mail we should consider the overall aspects of air emissions treatment.

An evaluation of the capability of the Phase 1A area SVE treatment system to meet permissible air emissions requires an evaluation of the VOC load in the extracted air. In turn, air loading depends on air flow rates and the VOC concentrations in the extracted air. Predicting either the field flow rate or the VOC concentrations in the vapor stream with great accuracy is not possible because we have not extracted air from the saturated deep till. Furthermore, the observed geometries of the multiple hydraulic fractures propagated from each fractured recovery well appear to be nonidealized. This complexity does not lend itself well to modeling predictions.

As you have correctly noted, we have limited data, especially from the deep till, upon which to base our design criteria. The best we can do is to develop a range by which the field conditions will be bounded. In our estimate of air flow rates and VOC loads presented in the May 30 letters, we proposed certain criteria. For the reasons noted previously, these criteria were developed using a highly conservative approach.

Specifically, our conservatively high estimates were based on the following assumptions:

- **VOC Concentrations**

  Initial vapor concentrations were assumed to be equal to maximum groundwater VOC concentrations, i.e., the vapor phase is in equilibrium with the liquid phase. We believe that this likely results in the overestimation of soil VOC concentrations by at least one order of magnitude.

- **Attenuation Factor**

  We showed, in our May 30 letter, that air loading rates were acceptable using a dilution attenuation factor of 0.01. This is an order of magnitude higher than the cited references suggest (0.001) is appropriate for steady state VOC mass transfer.

- **Air Flow Rates**

  Computer simulations using permeabilities up to an order of magnitude higher for the lower till were used to develop an air flow rate estimate. Additionally, the 40 scfm flow rate used includes a contingency factor added to the simulation estimate. Please note that this estimate is used for the time when the unconsolidated till deposits are fully dewatered. This state will be achieved or approached only after a significant period of groundwater extraction (6-12 months minimum). Initially, air flow rates will be much lower.

During previous discussions including you and Dom DiGiulio we have recognized that the rate of diffusion will limit VOC mass removal. Conceptually, the most efficient air flow rate would be equal to the rate of diffusion. Despite decreased efficiency, the SVE system will be operated at a much higher flow rate. The actual flow rate will be governed by the permeability of the Phase 1A area soils.

Initially, the system flow rate and VOC mass loading are expected to be much lower than the estimates we have provided. These estimates were developed to represent the anticipated period of highest mass loading, which is the period before attainment of steady state (dewatered) conditions. During this period, dewatering will progress toward steady state conditions while the system flow rate is expected to approach the estimated 40 scfm average. With time greater volumes of unsaturated, VOC-contaminated soil progressively will become accessible for vapor extraction, thereby resulting in the maximum VOC concentrations in the extracted air stream.

The estimates of VOC air stream concentrations and air flow rates provided are ultraconservative. Due to the conservative approach employed, our estimates reflect the worst case for both inputs, that is, high air flow rates with high VOC concentrations. This represents a mutually exclusive condition. At air flow rates higher than expected, mass removal will be less efficient and the VOC concentrations in the extracted air will be much lower than predicted in our May 30 letter. Based on the approach used to estimate VOC mass loads, the potential for VOC concentrations to approach, much less, exceed our estimate is extremely limited.

Based on the estimated air loading rate developed using the conservative approach we have presented, the actual VOC loading rate reasonably can be expected to be one to two orders of magnitude lower than our summary estimate.

DEP COMMENTS - Air Flow Rates

1. *Is a printout of the simulation (input and output) available? If feasible, please provide.*

The software AIR3D was used in the simulation. We extracted data directly from the screen. Pressure profiles are attached.

2. *What fracture geometry(ies) was simulated? (e.g., diameter, thickness, number of fractures, depths of fractures).*

The data used in the simulation are as follows:

Soil permeability: $k_x = k_y = k_z = 2.89 \times 10^{-9}\ cm^2$
$3.11 \times 10^{-12}\ ft^2$

Fracture permeability: $k_x = k_y = 100 \times k_{soil} = 3.11 \times 10^{-10}\ ft^2$
Sand permeability $\approx 5.4 \times 10^{-10}\ ft^2$
Sand $k_z = 4 \times 10^{-12}\ ft^2$

The fracture thickness was very conservatively assumed to be 0.5 feet. The actual fracture is about 0.02 feet thick. The permeability should be 0.02/0.5 times the value used. That is, we have a safety factor of 25 in the fracture geometry to limit the fracture pressure drop effects.

Fracture radius = 17 feet

Top fracture modeled at 8.5 feet deep
Second fracture modeled at 12 feet deep

The flow rates from the model were corrected for the field 12 foot and 19.5 foot deep fractures as follows. The modeled rate for the top fracture (12 feet deep) was multiplied by 8.5/12. The modeled rate for the modeled 12 foot deep fracture was used for the deeper fractures (19.5 feet).

3. *Did the model assume that all fractures were at 12 inches mercury vacuum at the well? Was there any simulation of the "push-pull" operation?*

All fractures were modeled at 12" Hg vacuum at the well. Also "push-pull" was simulated.

4. *Please provide a radial vacuum and air flow profiles to clarify. "... appreciable radial pressure drops within the fractures... limited the vacuum force for low." (End of para. 1).*

See the attached figures for pressure drop profiles with radial distance.

5. *Please itemize the nineteen fractures that you consider were created at the six wells.*

Created Fractures:

Well I.D.	Number of Fractures
A	4
E	4
G	3
H	2
I	4
J	2

6. *Please explain "With the elimination of the identified surface vents, we do not expect that fracture interconnection, regardless of degree, will lead to an increase in air flow recovery rates." (Page 2, second paragraph). Does this refer to on vacuum? (In "push-pull" operation, shunts between fracture would clearly require increased flow rates.)*

The statement refers to total vacuum case. If, "push-pull" case is used, then a shunt between fractures could result in increased flow. If shunt is present then "push-pull" is not a viable system to use, because most of the air would be short circuited between fractures.

DEP COMMENTS - VOC Loads

1. *Please provide a copy of the second reference.*

A copy of the Rodriquez-Maroto reference was sent to DEP on June 3.

2. *Maximum VOC loads are based on 40 scfm per well. This use of this rate must be resolved.*

The comments preceding this portion of this letter address air flow rates.

3. *It is important to repeat the basis for applying the dilution attenuation factors to the equilibrium vapor concentration in your application to the Air Bureau.*

Noted. We will include the discussion.

4. *If the combined effect of total air rate and air distribution is such that adequate air circulation does not reach significant potions of the till matrix, then VOC removal rate could be affected due to the low concentration gradients.*

Low concentration gradients produce low diffusion rates. This is a possibility especially due to the nature of the till. Low diffusion rates would support low attenuation factors. We have suggested this condition via the literature sources cited.

5. *The maximum load estimates should allow the wells to be operated in the most efficient fashion; throttling to reduce VOC load limits should never be necessary.*

A properly designed remedial system accounts for as many contingencies as cost-effectively possible, including the reduction of flow rates for any number of reasons. If the maximum air loadings were approached, the site remediation efforts likely would be complete in only a few years. We do not think this will happen. Even if reducing flow rates for a short time was necessary, the effect on the duration of the operation of the system would be insignificant. In many respects we would like to see high VOC concentrations in the vapor stream. Not only will it shorten the system operation, it will result in much more efficient carbon filtration. Rather than throttling air flow rates due to high VOC concentrations, the throttling probably would occur because the vapor-phase VOC concentration will be too low.

We hope these responses clarify the intent of the design criteria. Once we have concurrence on these issues we can prepare the appropriate air permit applications.

Sincerely,

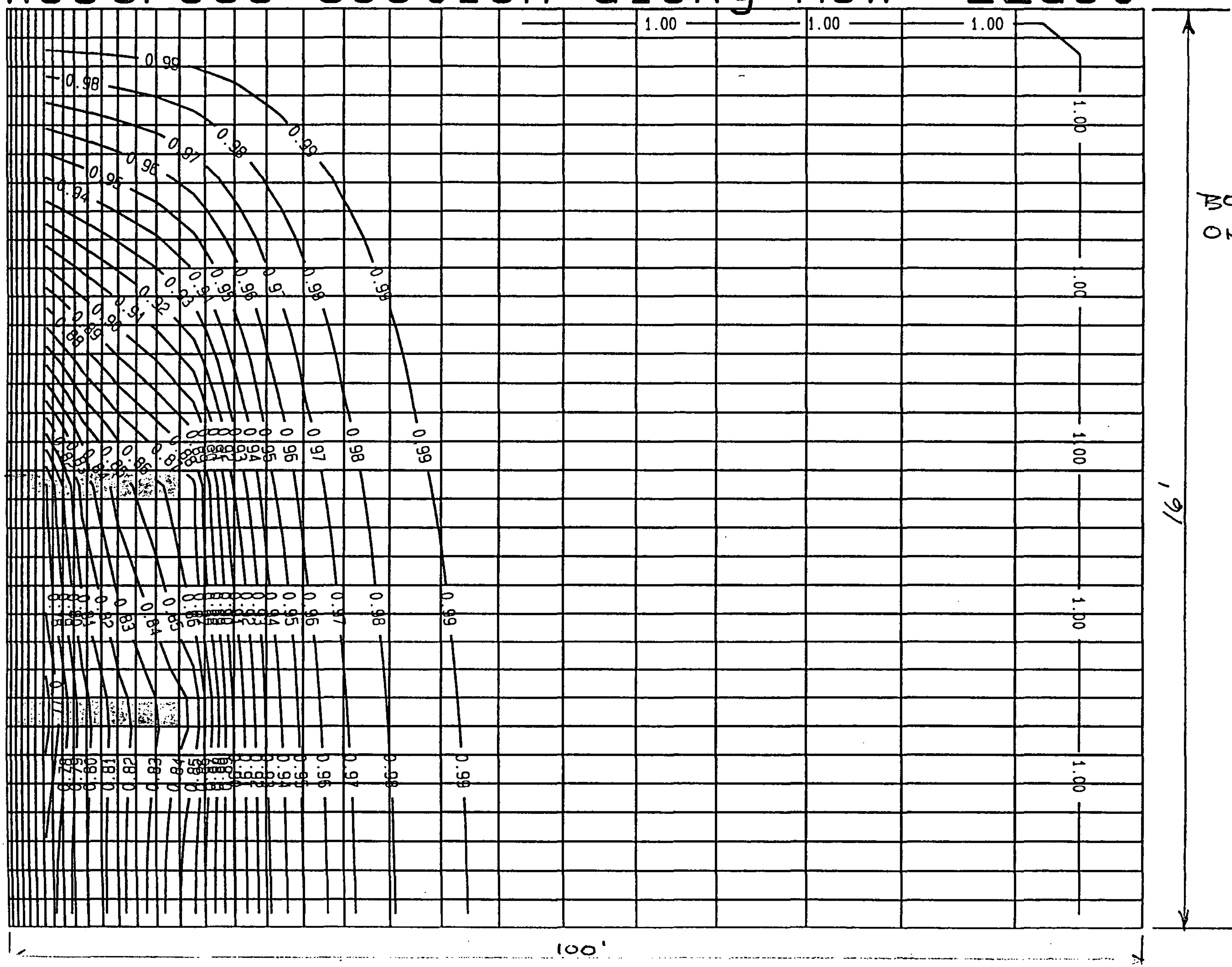
David L Bramley

David L. Bramley, P.E.
Project Manager

Enclosure

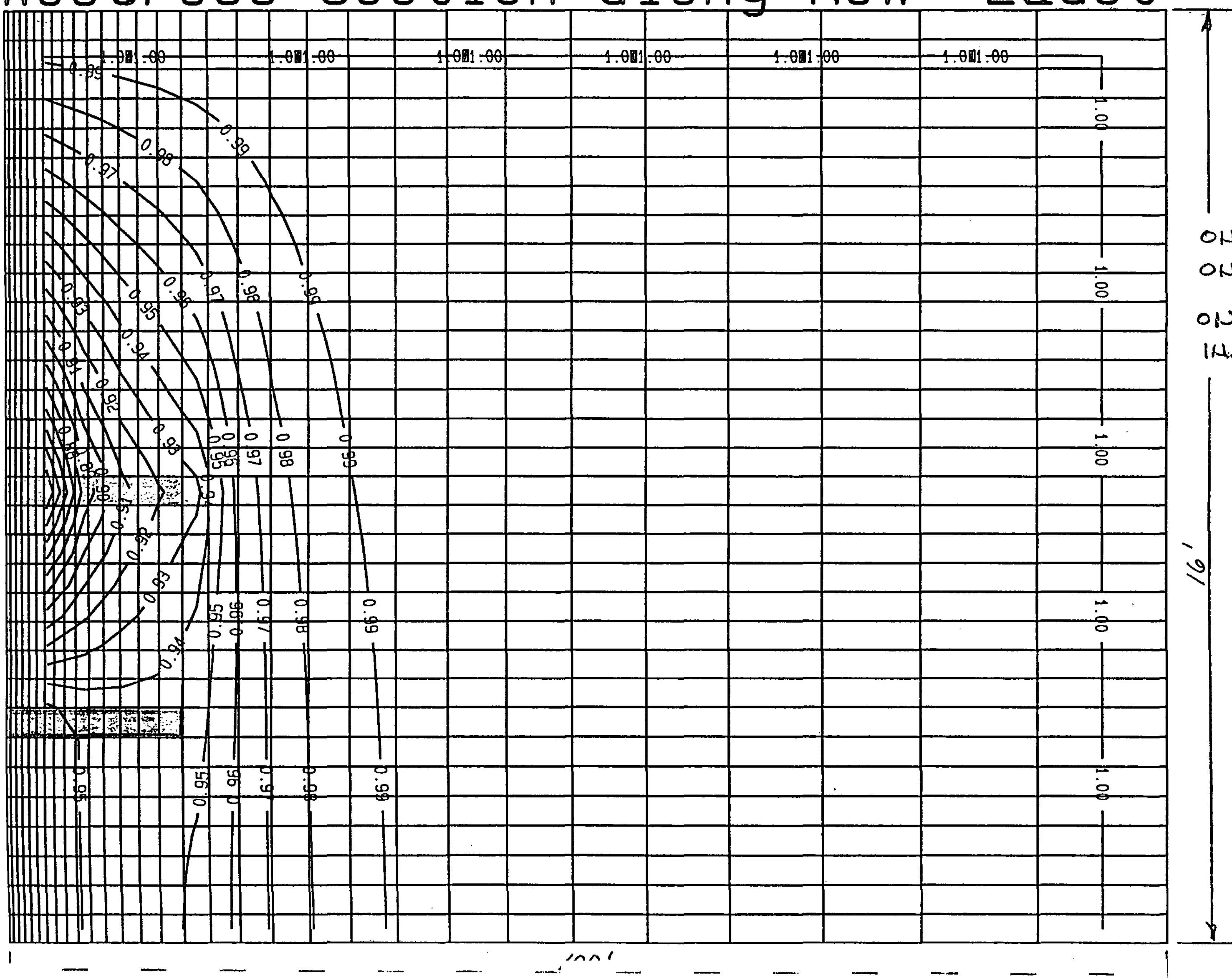
c. w/encl. Gary Kennett - Linemaster
Elise Jakabházy - EPA
Cinthia McLane - M&E
Mike Marley

West Cross-Section along Row 29 East



BOTH FRACTURES
ON VACUUM

West Cross-Section along Row 29 East



ONE FRACTURE
ON VACUUM

ONE ON ATMOSPHERIC
INJECTION

Fuss & O'Neill Inc. *Consulting Engineers*

146 Hartford Road, Manchester, CT 06040-5921
TEL 860 646-2469 FAX 860 643-6313

1200 Converse Street, Longmeadow, MA 01106-1721
TEL 413 567-9886 FAX 413 567-8936

Providence, RI TEL 401 828-3510

October 7, 1997

Mr. Martin Beskind
Environmental Analyst
Bureau of Waste Management
DEP-PERD
79 Elm Street
Hartford, CT 06016-5127

RE: Linemaster Switch Corp.
SVE Air Flow Modeling

Dear Mr. Beskind:

Previously you asked about friction losses due to air flow in the fractures. We originally used a six-inch thick fracture with a fracture permeability of 3.11×10^{-10} ft^2 and a sand permeability of 5.4×10^{-10} ft^2 or 100 times the permeability of the soil. We discussed friction losses in the fractures with Larry Murdoch and increased the permeability of the sand to 1,000 times the permeability of the soil. We also calculated the effective horizontal and vertical permeabilities of the fractures based on a fracture thickness of 0.02 feet in a vertical simulation grid of 0.5 feet. A copy of the calculations is attached. The result of the calculations is that the permeability for the 0.5 foot grid simulation is approximately 2½ times less than the previous simulation with the lower permeability and thicker fracture. The air flow is reduce from approximately 36 scfm/well to approximately 21 scfm/well using the revised permeabilities.

The vacuum at the end of the fracture when vacuum is applied to both fractures is 0.93 atmospheres when the vacuum applied to the well is 0.6 atmospheres (12" Hg). Therefore, if would appear from the simulation that there is the potential for appreciable vacuum loss in the fractures, and this would limit the air flow out of the fractures.

In practice, this resistance may be mitigated by the higher soil permeability at shallower depths and by naturally occurring fractures in the soil. Nevertheless, friction losses in the fractures should be considered in the prediction of air flow from the wells.

86088\A7\DLB0924A.WPD
Corres.

We hope this information will address the issue to your satisfaction. If you have questions please call me or Herb Klei.

Sincerely,

David L. Bramley

David L. Bramley, PE, LEP
Project Manager

Enclosure

c. w/encl. Gary Kennett - Linemaster
Elise Jakabházy - EPA
Cinthia McLane - M&E
Mike Marley

86088\A7\DLB0924A.WPD
Corres.

LINEMASTER PRESSURE DROP IN FRACTURES

Soil Permeabilities:

$\bar{K}$ of soil @ 2.9×10^{-9} cm^2 = 3.12×10^{-12} ft^2

K_H (sand) = 1000 K_H soil = 2.9×10^{-6} cm^2
Assume fracture is 0.02 ft. thick

$$K_H\ fracture = 2.9\ X\ 10^{-9}\ X\ \frac{.48}{.50} + 2.9\ X\ 10^{-6}\ X\ .02/.50$$

$$= 2.78\ X\ 10^{-9} + 1.16\ X\ 10^{-7}$$

$$= 1.18\ X\ 10^{-7}\ cm^2 = 1.27\ X\ 10^{-10}\ ft^2$$

$$K_V\ fracture = \frac{1\ X\ \Delta Z}{\Delta Z_1/_{K_1} + \Delta Z_2/_{K_2}}$$

$$\frac{1\ X\ 0.05}{.02/2.9\ X\ 10^{-6} + .48/2.9\ X\ 10^{-9}}$$

$$= \frac{0.50}{6.80\ X\ 10^3 + .1655\ X\ 10^9}$$

$$= \frac{.50}{.1655\ X\ 10^9} = 3.02\ X\ 10^{-9}\ cm^2 = 3.25\ X\ 10^{-12}\ ft^2$$

Conditions:
12" Vacuum of Layer 17
Vac/Atm. At Layer 25
Fracture Radius = 17'

VACUUM/VACUUM CASE

	VOL (cfs)	MASS (lb/sec)
OVERALL		
Output from well	-0.143875 acfs	-0.0066804
Inflow from land surface	0.0862681 scfs	0.00667599
LAYER 17		
Inflow to top of fracture	0.0859738	0.00607607
Flow through bottom of fracture	-0.03486	-0.00259139
Flow from layer	-0.0757013	-0.00351496
LAYER 25		
Inflow to top of fracture	0.0407836	0.00283823
Flow through bottom of fracture	0.00548514	0.000288044
Flow from layer	-0.0681738	-0.00316544

Flow from well = 0.0862681 cfs X 60 sec/min X 4 quarters/well = 20.7 scfm

ΔP file saved as PIUN.F3Q.PLT

VACUUM/ATMOSPHERE CASE

	VOL (cfs)	MASS(lb/sec)
Layer 17 (Vacuum)		
Flow into top of fracture	0.0465885	0.00333972
Bottom into bottom of fracture	0.0141435	0.000922031
Flow from fracture	-0.0919749	- 0.00427057
Layer 25 (Atmosphere)		
Flow into top of fracture	-0.0118406	-0.00087751
Bottom	0.00151237	0.000107236
Flow from fracture	0.00976419	0.000755617
OVERALL		
Surface	0.0453961	0.00351304

Output from well = 0.0919749 cfs @ 12" VAC (for ¼ diameter)

$$\text{Well output (scfm)} = 0.0919749 \text{ cfs } X \text{ } 60 \text{ sec/min } X \frac{17.92}{29.92} X \text{ } 4$$

$$= 13.22 \text{ scfm}$$

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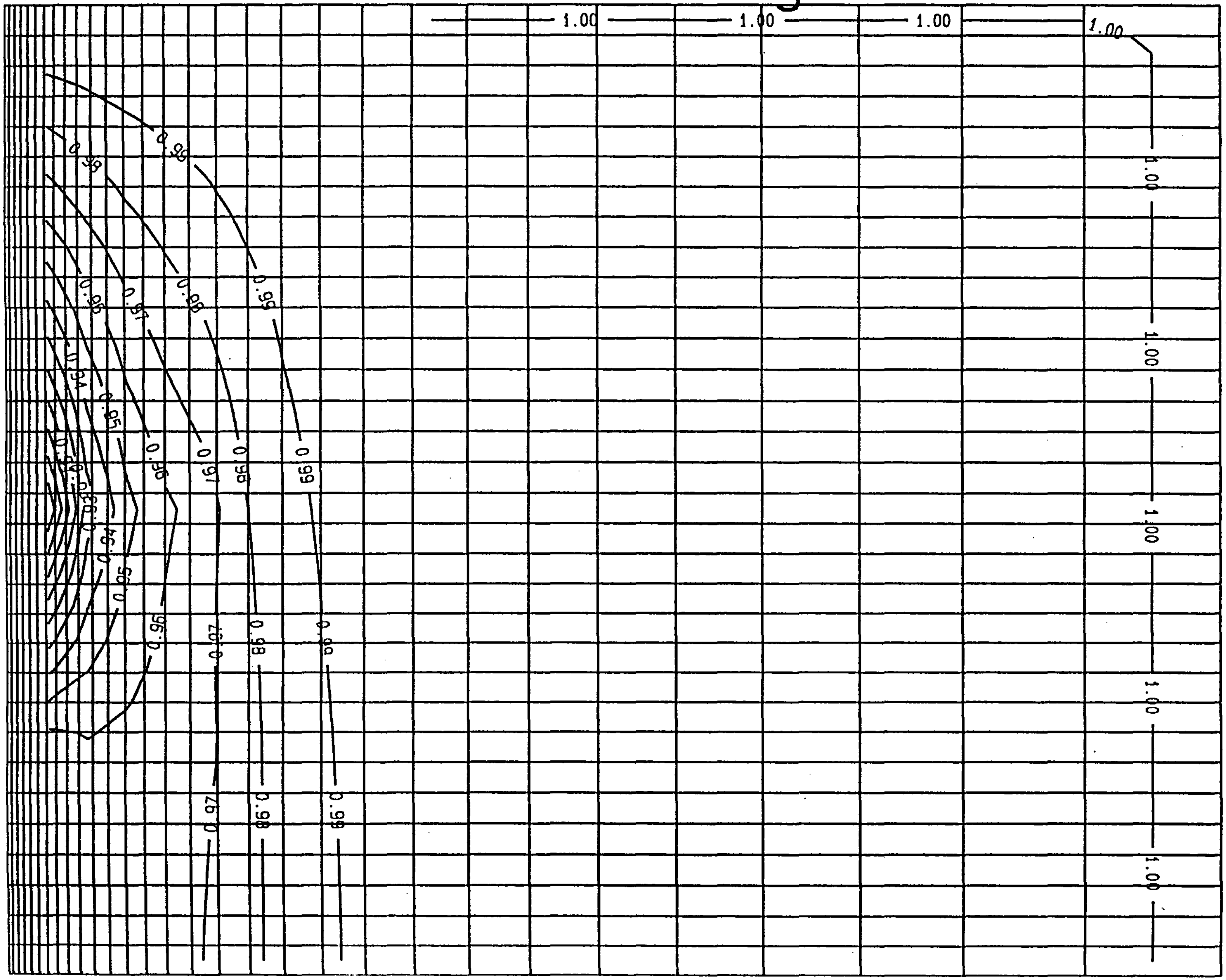
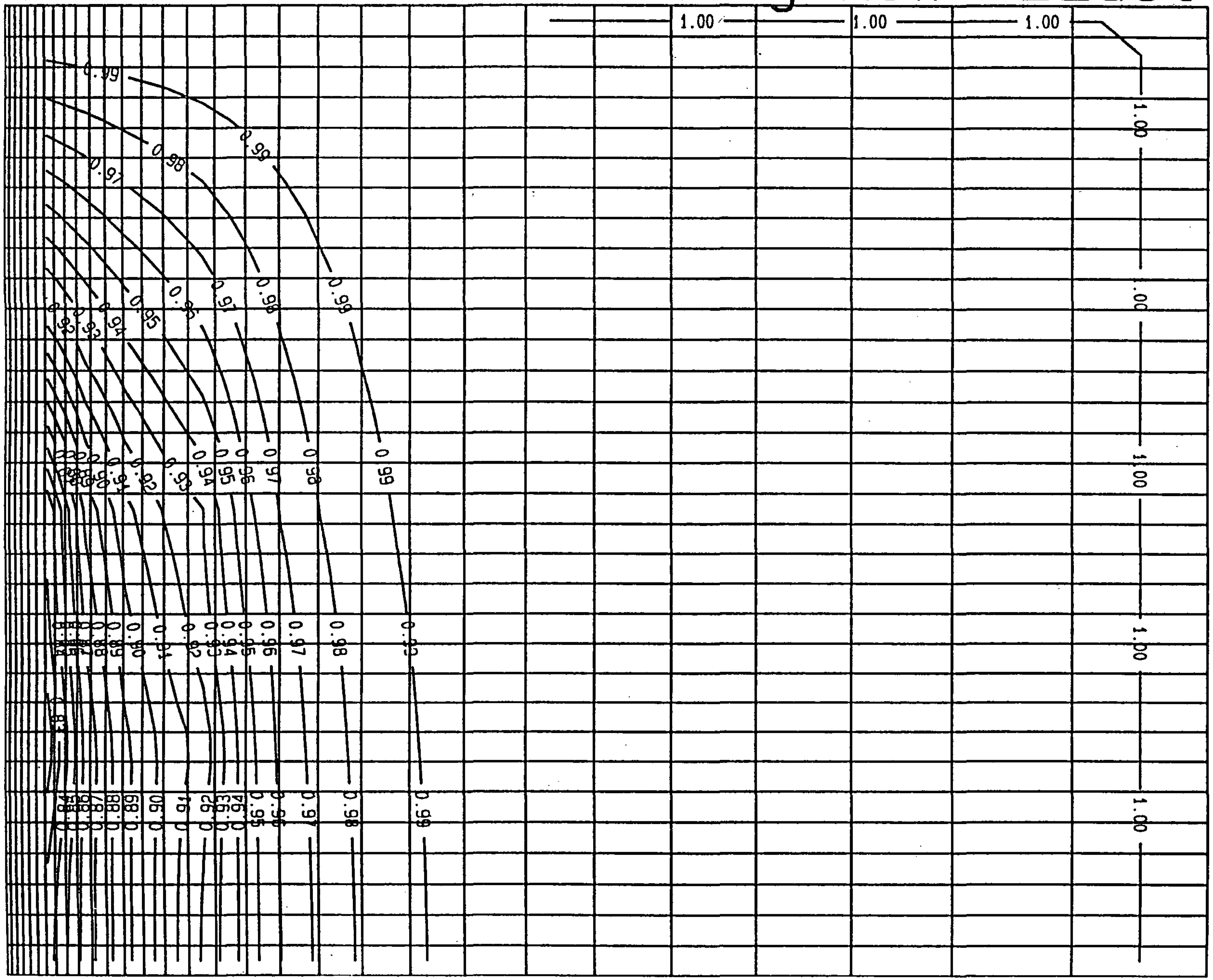
West Cross-Section along Row 29 East



Cross-Section along Row 29

West East



APPENDIX H

APPENDIX H

SVE VOC CONCENTRATION DATA

Fuss & O'Neill Inc. *Consulting Engineers*

146 Hartford Road, Manchester, CT 06040-5921
TEL 860 646-2469 FAX 860 643-6313
INTERNET: www.FandO.com

Other Offices:
Longmeadow, Massachusetts
Fairfield, Connecticut
East Providence, Rhode Island

December 4, 1997

Mr. Jaimeson A. Sinclair
Bureau of Air Management
Engineering and Enforcement Section-Permitting Group
79 Elm Street
Hartford, Connecticut 06106

RE: Air Permit Application
Linemaster Switch Corporation

Dear Mr. Jaimeson:

In response to your question the following will explain how the quantities were calculated in Table 1 (Appendix B) of the SVE permit application.

Equivalent Vapor Concentration (ug/l)

[Henry's Constant] * [VOC conc. in groundwater] * [1,000 l/m^3] ÷ [Ideal Gas Constant (0.082 $\frac{\text{l - atm.}}{\text{°K- mol}}$) * 288° K (15° C)]

Average TCE conc. $\frac{0.0053 * 124{,}391 \text{ ug/l} * 1{,}000 \text{ l/m}^3}{0.082 * 288} = 28{,}285$ ug/l

Air Loading (lb/day)

[Air flow from wells (280 cfm)] * [Equivalent vapor conc.]*[28.32 l/cf]*[1,440 min/day] ÷ [454 gm/lb * 10^6 ug/gm]

Average TCE load $\frac{280 \text{ cfm} * 28{,}285 \text{ ug/l} * 28.32 \text{ l/cf} * 1{,}440 \text{ min/day}}{454 \text{ gm/lb} * 10^6 \text{ ug/l}} = 711.4$ lb/day

We hope this clarifies the calculations. If you have any questions, please do not hesitate to call.

Sincerely,

David L. Bramley

David L. Bramley, PE, LEP
Project Manager

86088\A7\DLB1204C.WPD
Corres.

TABLE 1

ESTIMATED VAPOR CONCENTRATIONS USING EQUILIBRIUM GROUNDWATER CONCENTRATIONS (ug/L)

TCE

DATE / WELL	3/5/97	3/21/97	4/18/97	5/2/97	MAX	AVERAGE
FW-A	9900	4700	6900	25000	25000	11625
FW-E	5400	16000	5900	11000	16000	9575
FW-F(MW-26t)				780000	780000	780000
FW-G	150	240	27000	44000	44000	17848
FW-H	7200	20000	15000	4800	20000	11750
FW-I	720	44000	41000	2900	44000	22155
FW-J	1100	240	62000	7800	62000	17785
Average					141571	124391
Henry's Law Constant (atm/mol/m^3)15oC		0.00537				
Equivalent Vapor Conc (ug/l)=					32191.7	28285.1
Air Loading (lb/day) =					809.7	711.4

c-1,2 DCE

DATE / WELL	3/5/97	3/21/97	4/18/97	5/2/97	MAX	AVERAGE
FW-A	1200	670	1700	3500	3500	1768
FW-E	1100	1600	1600	5400	5400	2425
FW-F(MW-26t)				2900	2900	2900
FW-G	6800	150	2300	2100	6800	2838
FW-H	8100	14000	13000	7500	14000	10650
FW-I	150	28000	19000	13000	28000	15038
FW-J	150	170	2700	4200	4200	1805
Average					9257	5346
Henry's Law Constant (atm/mol/m^3)15oC			0.0023			
Equivalent Vapor Conc (ug/l)=					901.6	520.7
Air Loading (lb/day) =					22.7	13.1

1,1,1 -TCA

DATE / WELL	3/5/97	3/21/97	4/18/97	5/2/97	MAX	AVERAGE
FW-A	10	2.5	25	100	100	34
FW-E	15	110	50	59	110	59
FW-F(MW-26t)				500	500	500
FW-G	150	10	100	100	150	90
FW-H	15	200	160	53	200	107
FW-I	150	660	710	1500	1500	755
FW-J	150	14	100	79	150	86
Average					387	233
Henry's Law Constant (atm/mol/m^3)15oC		0.0103				
Equivalent Vapor Conc (ug/l)=					168.9	101.6
Air Loading (lb/day) =					4.2	2.6

PCE

DATE / WELL	3/5/97	3/21/97	4/18/97	5/2/97	MAX	AVERAGE
FW-A	81	57	37.5	150	150	81
FW-E	120	100	75	37.5	120	83
FW-F(MW-26t)				750	750	750
FW-G	150	3.75	150	150	150	113
FW-H	150	110	100	37.5	150	99
FW-I	150	660	710	550	710	518
FW-J	150	1.9	150	99	150	100
Average					311	249
Henry's Law Constant (atm/mol/m^3)15oC			0.00979			
Equivalent Vapor Conc (ug/l)=					129.1	103.3
Air Loading (lb/day) =					3.2	2.6

Toluene

DATE / WELL	3/5/97	3/21/97	4/18/97	5/2/97	MAX	AVERAGE
FW-A	15	3.75	320	150	320	122
FW-E	2800	8500	2300	2900	8500	4125
FW-F(MW-26t)				750	750	750
FW-G	750	3.75	760	980	980	623
FW-H	3300	4300	4800	1500	4800	3475
FW-I	150	8900	9800	4200	9800	5763
FW-J	150	2.4	150	290	290	148
Average					3634	2144
Henry's Law Constant (atm/mol/m^3)15oC		0.00468				
Equivalent Vapor Conc (ug/l)=					720.2	424.8
Air Loading (lb/day) =					18.1	10.7

Ethylbenzene

DATE / WELL	3/5/97	3/21/97	4/18/97	5/2/97	MAX	AVERAGE
FW-A	10	2.5	25	100	100.0	34.4
FW-E	610	2000	260	270	2000.0	785.0
FW-F(MW-26t)				500	500.0	500.0
FW-G	150	2.5	100	100	150.0	88.1
FW-H	750	1300	660	79	1300.0	697.3
FW-I	150	1100	590	790	1100.0	657.5
FW-J	150	0.5	100	34	150.0	71.1
Average					757.1	404.8
Henry's Law Constant (atm/mol/m^3)15oC			0.00593			
Equivalent Vapor Conc (ug/l)=					190.1	101.6
Air Loading (lb/day) =					4.8	2.6

Xylenes

DATE / WELL	3/5/97	3/21/97	4/18/97	5/2/97	MAX	AVERAGE
FW-A	38	2.5	25	100	100.0	41.4
FW-E	2400	7100	1500	890	7100.0	2972.5
FW-F(MW-26t)				500	500.0	500.0
FW-G	150	2.5	100	100	150.0	88.1
FW-H	2600	4300	2400	520	4300.0	2455.0
FW-I	150	5200	2900	3900	5200.0	3037.5
FW-J	150	2.6	100	96	150.0	87.2
Average					2500.0	1311.7
Henry's Law Constant (atm/mol/m^3)15oC		0.00447				
Equivalent Vapor Conc (ug/l)=					473.2	248.3
Air Loading (lb/day) =					11.9	6.2

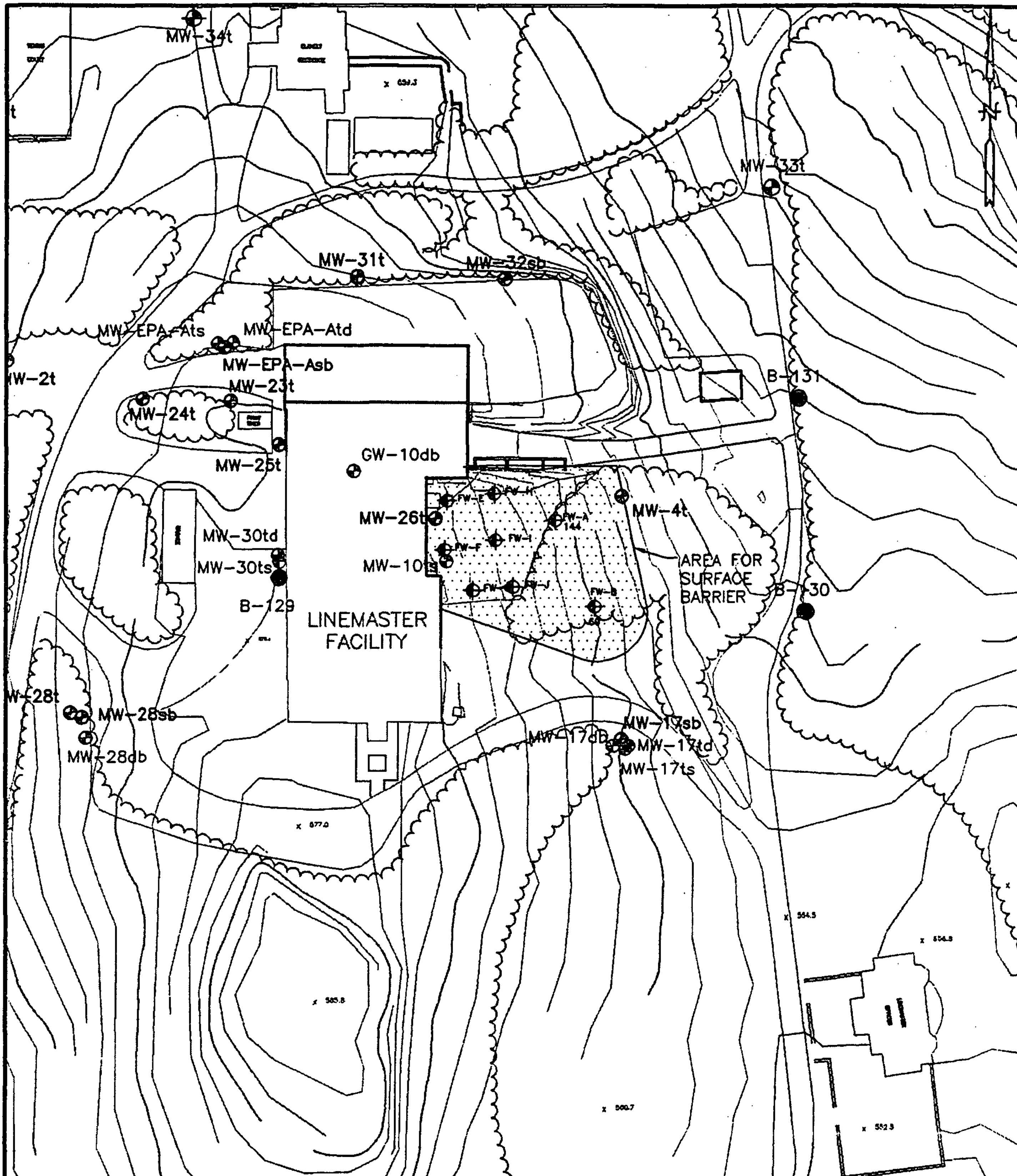
AIR LOADING CALCULATIONS

		AVERAGE	MAX
Total VOCs at Equilibrium (lbs/day)		749.1	874.6
Expected Loading w/ Diffusion Attenuation Factor (DAF)			
DAF =	0.001		
	lbs/day	0.7491	0.8746
	tons/year	0.1367	0.1596
DAF =	0.01		
	lbs/day	7.49	8.75
	tons/year	1.37	1.60

APPENDIX I

APPENDIX I

CAP SKETCHES



-100 0 100

SCALE: 1" = 100'

B-130 EXISTING BORING

MW-33t EXISTING MONITORING WELL

FW-B FRACTURED WELL

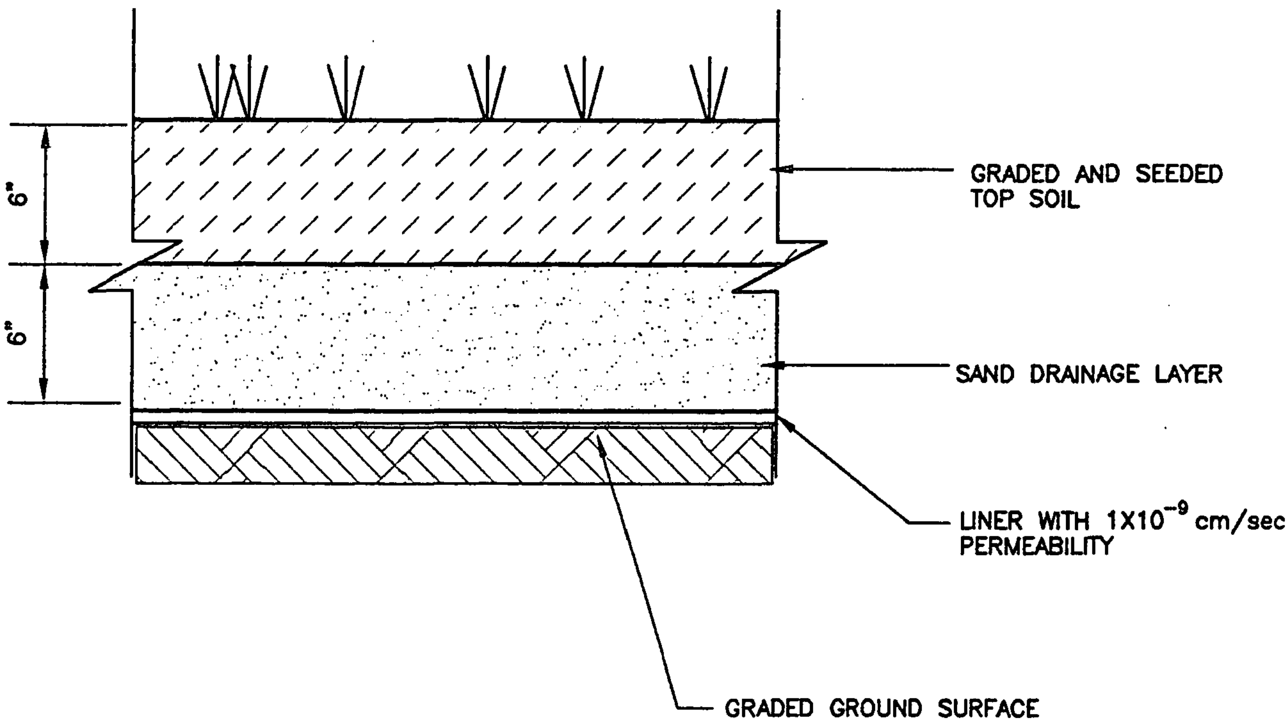
FN: A7\86088PBW | MS: S100 | PPP: ZONE1 | UCS: WRLD

FUSS & O'NEILL INC. Consulting Engineers
146 HARTFORD ROAD, MANCHESTER, CONNECTICUT 06040
(203) 646-2469

PHASE 1A SURFACE BARRIER
LINEMASTER SWITCH CORP.

PLAINE HILL RD. WOODSTOCK, CT.

PROJ. NO.: 86-088A7 DATE: FEB. 1998 SCALE: 1"=100'



FN: A7\86088DS
MS:
UCS:
PPP:

FUSS & O'NEILL INC. Consulting Engineers
146 HARTFORD ROAD, MANCHESTER, CONNECTICUT 06040
(203) 646-2469

PHASE 1A SURFACE BARRIER

TYPICAL CROSS-SECTION

LINEMASTER SWITCH CORP

PLAINE HILL RD. WOODSTOCK, CT.

PROJ. NO.: 86-088A7 DATE: FEB. 1998 SCALE: N.T.S.

APPENDIX J

APPENDIX J

SOIL MOISTURE CONTENT MONITORING (TDR) MEMORANDUM

TECHNICAL MEMORANDUM

TO: Elise Jakabházy, RPM

FROM: David Bramley, PM
Tim Whiting

DATE: March 19, 1997

RE: Linemaster Switch Corporation
Soil Moisture Content Monitoring

COPY: Gary Kennett - Linemaster
Martin Beskind - DEP
Cinthia McLane - M&E
Mike Marley - Envirogen

Soil moisture content is an important parameter that will be monitored during operation of the Linemaster Phase 1A DVE system. This memo provides a brief description of methods used for determining soil moisture, their advantages and limitations, suitability of the method to the intended site application, and relative costs. After substantial consideration, an in-situ time domain reflectometry (TDR) method is recommended for the Linemaster Switch Corp site.

PURPOSE FOR SOIL MOISTURE CONTENT MONITORING

Knowledge of soil moisture content is critical to determining whether air flow may occur in certain volumes or intervals of subsurface soils. Although air flow physically may be possible due to low moisture contents, vapor recovery is dependent upon the spatial relationship between that volume of soil and existing hydraulic fractures.

Although the most substantial mass of contaminants is present below the water table, the system planned for installation in the Linemaster Phase 1A area has been designed to optimize mass removal from the vapor phase. Vapor phase mass removal requires that the saturated unconsolidated till deposits be dewatered. As demonstrated during the 1995 pilot test, dewatering by hydraulically fractured recovery wells will draw down or depress the water table. However, substantial moisture may remain in the soils for an extended period of time following the depression of the water table. Only when the soil moisture content is sufficiently reduced will air be able to flow through large sections of the unconsolidated deposits. Until then, vapor phase mass removal is not optimized. The monitoring of soil moisture content in the Linemaster soils will enable us to assess progress in reducing soil moisture content and make recommendations for optimal system operation.

SITE REQUIREMENTS

The following requirements have been developed to identify the soil moisture content determination method best suited to the Linemaster application. The measurement of temporal change at fixed locations will most effectively assess soil moisture content during the dewatering the Phase 1A area. Therefore, in-situ monitoring methods are strongly preferred. Additionally, the chosen method must be:

- well established, well documented, and cost effective
- capable of determining soil moisture content at depths up to 50 feet below grade
- capable of accurately determining volumetric soil moisture content over the wide range of expected values (5% to 40%).

EXISTING METHODS

A review of existing methods to measure soil moisture content has indicated that the two best suited methods are neutron probe logging and time domain reflectometry. Each of these methods has been considered in detail. The following sections provide a brief description of the theory, history of application, and discussion of site suitability for each method. Table 1 presents a summary of advantages and limitations of each method.

NEUTRON PROBE LOGGING

Theory

All neutron logging devices operate by measuring the change in energy of neutrons that pass through the formation. Neutrons emitted from the tool's source lose energy primarily through collisions with hydrogen atoms. These hydrogens are usually in the form of free water in pore spaces, water bound to formation minerals and water between the borehole wall and the tool's source and detectors. In cased hole logging, hydrogen in the casing material (PVC) and water remaining in the grout also will influence the measurement. The effects of water between the tool's source and detectors and the casing wall are minimized by pressing the tool against the inside of the casing. To a lesser extent, the presence of nuclei that tend to capture low energy neutrons, such as chlorine and boron, also will influence the measurement by reducing the number of neutrons counted by the detector. The strength of the neutron source and the initial energy of the neutrons determines the distance that the neutrons penetrate the formation ("depth of investigation") and the number of neutrons that are counted by the detector. A stronger source provides higher counts and is desirable for improved statistical evaluation and repeatability of the measurement.

TABLE 1

COMPARISON OF METHODS

ADVANTAGES AND DISADVANTAGES	NEUTRON LOGGING	TDR
Established/Accepted methodology	Yes	Yes
Method widely used for soil moisture measurement	Yes (construction and highway applications)	Yes (agricultural and soil physics)
Specialized equipment and service provider	Yes (both)	Yes (equipment) No (provider)
NRC license/training required	Yes, source is radioactive	No
Service provider availability	Limited	Unlimited after training
Results	Qualitative	Quantitative
Initial equipment costs	None	High
Subsequent monitoring costs	Very High	Low
Data Analysis	Specialized	Can be automated

Application History

To our knowledge, neutron logging has not been extensively used for applications similar to those required at the Linemaster site. This section primarily has been developed based on information provided by Weston Geophysical. Weston has performed neutron logging to evaluate soil moisture content changes above and below the water table at two sites.

Well construction is critical for neutron soil moisture logging. The well materials can mask formation soil moisture readings. With periodic logging events, casing/annulus conditions should be maintained as constant as possible (i.e. borehole logged only when completely dry or filled with water) and theoretically should vary only as a function of moisture content change.

Weston stressed that the results provided from the logging would be qualitative, not quantitative. The neutron detectors record returning neutrons in counts per second. To have a quantitative soil moisture measurement, a given number of counts per second must be correlated to a specific soil moisture content. Corrections also must be estimated for the effects of the well casing material and grouted annulus. To make these correlations, a model of the well and the adjacent stratigraphic units present at various degrees of saturation would have to be constructed and logged by the tool to develop a calibration curve and borehole corrections. Even with calibration soil samples, Weston indicated that it would be extremely difficult and time consuming to create accurate models that duplicate actual logging conditions. Therefore, the soil moisture measurements recorded by neutron logging would be considered qualitative.

To evaluate qualitative changes in soil moisture, it is imperative that a baseline log be established before dewatering and that consensus is achieved regarding what that baseline represents (e.g. 100% saturation/moisture content below the water table). Subsequent logs would be compared to the baseline with the assumption that all parameters affecting the log are constant with the exception of soil moisture content. With these assumptions, changes in the counts received by the tool can be attributed to changes in soil moisture content and a percent change in soil moisture from the baseline can be calculated. Weston indicated that this comparison and "calibration" to the assumed baseline conditions and subsequent evaluation of the logging results could be done without the initial collection and lab analysis of soil samples for moisture content. However, they suggested that it would be prudent to collect and analyze some soil samples to confirm the correlation between the baseline soil moisture assumptions and actual soil moisture.

Casing Installation

The neutron logging device cannot distinguish between water in the annular fill material and moisture found in the adjacent undisturbed formation. Therefore, ideally, neutron logging is performed from inside a thin-walled casing, typically aluminum, that is driven into place to avoid creation of an annular space.

Weston indicated that, as a minimum, a two-inch diameter casing was required to accommodate their equipment. The casing material may be aluminum, steel, or PVC. Sonic drilling techniques appear to be the only reliable means of installing a casing without the creation of an annular space. Recent Pine & Swallow two-inch diameter casing test installation (vibratory direct-push) results would not be satisfactory for this purpose.

Conventional drilling methods can be used to install casings with annular spaces for neutron probe logging. Weston stated that the casing diameter is not important (as long as it could accommodate their tooling). The critical objective is to minimize the size of the annulus. Weston agreed that a technique similar to the installation of the FW-A and FW-B fractured recovery wells could be used to install a neutron probe casing with an annulus of slightly less than one inch surrounding the casing. Alternatively, a four-inch steel casing could be installed using drive and wash methods. As the base of the casing would be open, this would require placement of a water-tight seal within the base of the casing. Regardless of the sealant, the long-term performance of the seal is uncertain.

Calibration Samples

Weston indicated that if soil samples are to be collected for analysis to help determine baseline moisture content, then it would be best to sample the different stratigraphic units present. At the Linemaster site, there are three hydrostratigraphic units: unsaturated upper till, saturated upper till, and saturated lower till. Two samples from each unit should be sufficient to calibrate the equipment. Sample collection would be performed using the California Modified split-spoon sampling technique or a derivation thereof. Essentially, this would entail the collection of soil samples in six-inch diameter tubes that subsequently would be sealed with teflon and caps.

Miscellaneous

Weston recommended initially logging baseline conditions over a period of two to three days to ensure annular and formation conditions were stable. Subsequently, monitoring would be performed on periodic basis; most frequent at the start of dewatering and becoming less frequent as the level of site dewatering slows and stabilizes.

Weston has the appropriate NRC licenses and training. They would have to investigate Connecticut requirements to determine if there is reciprocity with other states, but it wouldn't be a major problem to become licensed in Connecticut if necessary. Weston assured us that the potential for losing the nuclear source in the casing was minimal. They noted that this has never happened in similar shallow cased wells during their ten plus years of experience.

TIME DOMAIN REFLECTOMETRY

Time Domain Reflectometry (TDR) is a remote sensing electrical measurement technique that has been used for many years to determine the spatial location and nature of various objects (Northwestern Univ., 1997). Radar is a familiar application of TDR that was developed in the 1930s. More recently, TDR methods have been developed for measuring soil moisture content. Its first application to soil water measurements was demonstrated by Davis (1975), Davis and Annan (1977), and Topp et. al. (1980). Since that time, TDR has become a well accepted method to make soil moisture measurements (Topp et. al., 1988) and has been used in the agricultural, soil physics, geotechnical, mining, and environmental fields. The American Society of Testing Materials (ASTM) began development of a standard TDR method for determining soil moisture content in 1994 (Attachment A).

Theory

Volumetric water content, as measured by TDR, is determined by measuring the travel time and attenuation of the amplitude of a very fast electromagnetic pulse propagated down and reflected back from the end of a transmission line imbedded in the soil. By determining the travel time in a transmission line of known length, the apparent dielectric constant of the medium can be calculated. The propagation velocity is a function of the soil bulk dielectric constant. As the dielectric constant of liquid water is approximately 81, compared to 1 for air and 3 to 5 for soil minerals, the soil bulk dielectric constant is influenced most strongly influenced by the presence of water. This large disparity between dielectric constants makes the TDR method relatively insensitive to soil texture and composition. Topp et. al. (1980) developed a mathematical expression describing the relationship between the apparent soil dielectric constant and volumetric soil water content. This empirical relationship may not apply to highly conductive soils or soils with large amounts of bound water or high organic matter content (i.e. peat deposits). In such cases, soil-specific calibration may be required to establish a unique soil dielectric constant/soil moisture content curve.

TDR has a reported accuracy to 1 or 2% of volumetric water content (Or, 1997) except near the upper and lower moisture content ranges (i.e. 100% and 0%). At soil moisture contents less than 5%, the dielectric constant of the soil minerals becomes dominant. Zegelin et. al. (1992) reported that TDR measurements are accurate to ± 10% of the **daily change** in soil water content.

Application History

The use of TDR to measure soil water content historically has been limited to shallow soils due to installation limitations. The commonly used TDR components include a testing device, which generates and records the electromagnetic signals, a probe that is buried in the soil to be tested, and coaxial cable lengths to link the tester and individual probes. With the use of today's technology, multiplexers and sophisticated data acquisition systems, continuous automated soil moisture monitoring using many probes is possible in remote locations.

Equipment

Until relatively recently, the equipment used for soil moisture content monitoring has been borrowed from other fields. This has presented somewhat of an operational problem because the instructions, documentation, or specifications for the involved components (cable tester, data logger, probe, cable, software) may have originated from as many as four or five unrelated sources companies.

The TDR tester most commonly used in the past is the Tektronix 1502B metallic cable tester. This piece of equipment was developed for testing metallic cables for shorts and faults. This tester still is used today. Specialty companies (e.g., Soilmoisture, Imko) have developed TDR testers specific to soil moisture measurement applications. The TDR tester records and stores the signal travel history and visually displays the resultant waveform on a built in oscilloscope or screen. The waveform data can be stored and downloaded for subsequent analysis using standard RS-232 communication hardware and software.

TDR probes used for soil moisture content measurements normally have two or three pins or rods. The pin length and diameter are designed for optimal signal transmission; typical lengths range from 15 to 30 cm with diameters in the 1/8" range. During installation, the pins can be subject to bending, which, if undetected, can result in inaccurate data. Pin breakage can occur if a rock is encountered during installation. Testing immediately after installation typically can identify these types of problems.

The probes are connected to the TDR tester using standard RG58 coaxial cable. Due to signal attenuation, the general maximum recommended length of this type of cable is 50 feet. Higher grade RG58 coaxial cable allows lengths of up to 60 m to be used for effective TDR signal transmission. A faulty or weakened connection between the probe and the cable can cause poor signal transmission. Again, testing immediately after installation can identify the existence of such a problem. Due to the limited history of application, the longevity of the TDR probes and connecting cables in the subsurface environment is uncertain.

Data Analysis

The analysis of the recorded TDR waveform formerly was performed manually using tangent matching techniques. Today, analytical software has been developed independently for commercial and/or public domain use. Developers include Waterloo (WAT-TDR), Utah State University (WIN-TDR), and Soilmoisture Equipment Corporation (WIN Trase). These software packages can be used to analyze waveform data from any cable/TDR tester that has RS-232 connections. TDR equipment designed for soil moisture applications (e.g., the Soilmoisture Trase system) commonly has built-in analytical capabilities and can provide real time soil moisture readings. Automated, periodic soil moisture readings are readily attainable using electronic data loggers. Although the software packages generally are user-friendly, as with any software package, an inexperienced user may unknowingly produce inaccurate data.

Probe Installation

Historically, TDR applications have been limited to depths of 5-10 feet. TDR probes were installed from the ground surface with the pins oriented vertically or with the probe pins oriented horizontally during trench sidewall installations. Or (pers. comm., 1997) reported a single probe installation in the base of a 20-foot deep borehole. Within the last year, FRx has developed proprietary equipment to install TDR and other like probes in the sidewall of an open eight-inch diameter borehole. FRx has installed more than 100 probes at depths up to 20 feet using this method. The depth to which a borehole will remain open in the site-specific soil type is the practical limitation on the depths that TDR probes may be installed using this new technique. Additionally, this method allows the installation of probes at multiple depths in the same borehole, thereby allowing detailed evaluation of vertical soil moisture content profiles. Borehole collapse would preclude the installation of probes in the collapsed interval.

RECOMMENDATION FOR LINEMASTER APPLICATION

After thorough review and consultation with experts in both fields, TDR has been determined to be the better technique for evaluating soil moisture content during DVE operations in the Phase 1A area at the Linemaster site.

Both methods are well established and documented and allow in-situ monitoring to assess temporal changes at fixed locations. However, TDR offers several distinct advantages over neutron logging as noted in Table 1. This method allows quantitative, volumetric soil moisture measurements and is more accurate over the entire range of expected soil moisture content (approx. 5% to 40%) without the use of a radioactive source. Additionally, TDR is significantly more cost effective than neutron probe logging for monitoring soil moisture content at the desired monthly frequency.

The following summarizes the proposed approach for using TDR to monitor soil moisture content in the Linemaster Phase 1A area. This approach has been developed by F&O with significant input from Envirogen, FRx, and Soil Moisture Equipment Corporation.

Locations

Consistent with the conceptual proposed monitoring plan, the TDR probes will be installed in two borings located in the vicinity of the FW-Fx casings. One TDR boring will be installed approximately five feet from the centralized casing locations to measure soil moisture changes in a densely fractured area. The other TDR boring will be installed near the limits of the FW-Fx casing fractures to assess soil moisture response in a less densely fractured area. Soil moisture changes in unfractured areas will be inferred based on the results observed near the fracture limits. TDR probe installations in an unfractured area likely would provide inconclusive results regarding the effectiveness of the remedial system at reducing soil moisture sufficiently to allow significant air flow and vapor recovery through the hydraulic fractures.

Testing Equipment

The Soil Moisture Equipment Corporation (SEC) Trase System has been specifically designed for soil moisture content measurement. This unit is a self-contained system capable of soil moisture measurement and can analyze and store TDR data and waveforms. Probe compatibility with the FRx installation equipment requires the use of TDR probes supplied by FRx. These 15-centimeter long probes and the attached RG58 coaxial cables have been inspected and modified as necessary by SEC to ensure compatibility with the Trase system. Automated analysis will be performed by the Trase system itself. Confirmatory analysis periodically may be performed using TDR software developed by SEC or others.

Probe Installation

Although TDR has primarily been used for shallow applications, FRx has successfully demonstrated the ability to install TDR probes at depths up to 20 feet below grade. Using the same technique, TDR probes can be installed at the depths (max. 45 ft) provided that the borehole remains stable. The TDR boreholes will be drilled using sonic drilling techniques, which will afford detailed hydraulic fracture spacing information. (Note: In the event that probe installation cannot be coordinated with the sonic drilling program, the probes will be installed at a later date in boreholes drilled using hollow-stem auger or air rotary drilling methods. In such case, the TDR boreholes would be located adjacent to locations where sonic drilling had confirmed fracture depths.)

Fifteen TDR probes will be installed in each borehole. The probes generally will be installed at five foot intervals. Probe locations relative to hydraulic fractures will be determined carefully and recorded. Modification of spacing will be made to allow installation at strategic spacing relative to observed hydraulic fractures. Conceptually, the probes will installed at least one foot distant from observed fractures. In one five-foot interval in the lower till (approx. 20 ft) in each borehole, TDR probes will be installed at approximately one-foot spacings (e.g. 20', 21', 22', 23', 24' and 25'). This close spacing will enable a detailed evaluation of soil moisture content change in the vicinity of hydraulic fractures. The high-density probe interval intentionally will be different in each borehole to evaluate soil moisture content change at depth.

Previous experience indicates that the till deposits are sufficiently stable to allow installation at most of the targeted intervals. Due to the presence of cobbles in the till, probe breakage can be expected to occur. A breakage rate of 20% has been allowed. Therefore up to 36 probes will be available for installation to provide 15 functional probes in each borehole.

Soil Moisture Measurement and Analysis

Initial soil moisture measurements will be made jointly by FRx and F&O. FRx will serve in the primary capacity and will demonstrate the appropriate techniques to F&O personnel. Immediately following installation, each TDR probe will be tested to ensure that the probe is functional and was not damaged during installation. If found to be damaged, additional probes will be installed at the same depth on the opposite side of the borehole or at a slightly different depth on the same side.

To confirm whether Linemaster soils are consistent with the established dielectric constant/soil moisture curve, six soil samples will be collected from each borehole during drilling and analyzed at a soil testing laboratory for volumetric soil moisture content. In each borehole, two soil samples will be collected from each of the three hydrostratigraphic units: unsaturated upper till, saturated upper till, and saturated lower till. The samples will be collected from the sonic drilling cores in two-inch diameter tubes. The tubes will be advance manually and filled as completely as possible. The ends will be sealed with teflon tape and capped to prevent moisture loss during transit.

The laboratory soil moisture results will be compared to soil moisture content values measured by TDR probes at equivalent depths. If found to deviate from the normal curve, these data can be used to create a site-specific curve suitable for future analysis.

Soil moisture measurements will be recorded approximately one to two weeks after completion of probe installation. Subsequently, soil moisture monitoring will be performed monthly. These measurements will be made by trained F&O personnel. FRx will assume a quality assurance role and review waveform data and analysis.

ATTACHMENT A

Development Document
ASTM Time Domain Reflectometry Soil Moisture Content Measurement

Standard Test Method for Determination of Water (Moisture) Content of Soil by the Time-Domain Reflectometry (TDR) Method

Subcommittee: D18.21.02
VADOSE ZONE MONITORING

Designation: DXXXX-94

Draft No: Draft 0

Date: June 21, 1994

* Note: This document was widely distributed at a recent technical conference.
TRW 3/31/97

Table of Contents

Standard Test Method for Determination of Water (Moisture) Content in Soil by the Time-Domain Reflectometry (TDR) Method

1.0 Scope

1.1 This test method covers the determination of water content (or moisture content) in soil by the use of the electromagnetic technique called Time-Domain Reflectometry (TDR).

1.2 This test method was written to detail the procedure for conventional TDR measurements of soil. Other TDR applications exist for the purpose of quantifying water content in soil and are not covered here, such as flat probe technologies and wetting front advance methods.

1.3 Commercial TDR applications exist which automate the TDR methodology and are not detailed in this test method. It is likely that overlap exists in the automated commercial systems versus this applied method, and the user is encouraged to adhere to this standard when applicable.

1.4 This standard is one of a series on vadose zone characterization methods. Other standards have been prepared on vadose zone characterization techniques.

1.5 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2.0 Referenced Documents

2.1 ASTM Standards

D653-90	Standard Terminology Relating to Soil, Rock, and Contained Fluids.
D18.21.89.09	Terminology for Vadose Zone (draft)
D18.21.89.16	Practice for Determination of Soil Moisture in Soil (draft)
D18.21.17	Guide for the application of Neutron Moderation in Pollution Investigation (draft)
D1452-80	Practice for Soil Investigations and Sampling by Auger Boring
D2216-80	Methods for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures.
D4643	Method for Determination of Water (Moisture) Content of Soil by the Microwave Oven Method
D4700-91	Guide for Soil Sampling from the Vadose Zone

D4944	Method for Field Determination of Water (Moisture) Content of Soil by the Calcium Carbide Gas Pressure Tester Method
D5220	Test Method for Water Content of Soil and Rock In-Place by Downhole Neutron Probe Method

3. Terminology

3.1 Time Domain Reflectometry (TDR) - an electromagnetic method for the determination of water content of soil.

3.2 Definitions of other terminology used in this guide may be found in ASTM Standard D653-90.

4.0 Summary of Test Method

4.1 A specially constructed, multi-wave guide TDR probe is inserted into the soil. The electronic cable tester (or automated commercial TDR electronics) is used to send a pulsed waveform to the probe. The cable tester then receives a return signal which was influenced by the dielectric constant of the soil, which in turn is a function of water content. An analysis of the waveform trace supplies the necessary information to calculate the water content of the soil.

5.0 Significance and Use

5.1 The determination of the water-content, or moisture content, of soil is one of the fundamental needs in the soil physics and hydrology disciplines. The need arises from requirements for defining the optimal time for irrigation, the infiltration rate, the soil-moisture flux, contaminant transport rates, and evaluating the potential for leakage from a waste site or a surface or subsurface barrier.

5.2 The TDR application covered in this test method is that used for point measurements of moisture content in soil. The application is either through manual insertion into the soil or by burying a probe in the subsurface to acquire moisture content data at a specific location. In addition, core samples may be tested with TDR at a drill site to acquire real-time soil moisture data.

6. Interferences

6.1 TDR measurements in conductive soils are hampered by the conductivity of the soil and the resulting signal attenuation. Typically, the amplitude of the voltage pulse reflected back to the TDR instrument is diminished in proportion to the soil's electrical conductivity. When the soil's electrical conductivity is high enough, there is insufficient signal strength for the TDR instrument to detect. TDR probes employing a balancing balun transformer are particularly susceptible to this effect. The balun transformer compounds the problems in analyzing the signals from probes with rod lengths of 15 centimeters or less [1].

6.2 Clay soils also attenuate a TDR probe signal. Conductive soils which have a significant amount of clay attenuate the signal the most. A partial solution to signal loss is to reduce the length of the probe [2]. However, as the probe length shrinks, the precision of the moisture content estimates worsens as rod lengths decrease.

6.3 A solution to the problem is to use a probe rod length of 15 centimeters and to electrically insulate the probe [3]. This can be accomplished by spraying the probe rods with a clear resin coating or applying a very thin layer of marine epoxy resin. The marine epoxy resin is a hard, non-conductive, and non-absorbing coating which adheres well to the metallic rods. The rods should be slightly abraded to enhance resin adhesion. The coating will have a minimal effect upon the accuracies observed.

6.4 Temperature effects have been observed when using TDR in the field. Temperature effects are particularly troublesome for systems where the user has predefined a probe beginning point within the software and employs long TDR probe cable lengths (~30 meters or more). The cable shrinks and contracts as a function of temperature. Naturally the maximum and minimum cable lengths occur during the warmest, and coolest times of the day, respectively. The solution is to avoid defining a beginning point of the cable tester trace within the users software. Also, thermal effects can be minimized by burying the cable or otherwise protecting the cable from exposure. In addition, the dielectric of the soil changes as a function of temperature.

6.5 A static charge on the coaxial cable may cause damage to the TDR cable tester unit. To avoid possible damage to the electronics, always dead-short the TDR probe leads to each other. This will discharge the static charge in the cable prior to connecting the cable assembly to the TDR cable tester unit.

7.0 Apparatus

7.1 The basic TDR system consists of a Tektronix[a] 1502B cable tester (or comparable unit) and a cable/probe assembly, as shown in Figure 1. The cable tester generates a fast rise time pulse which propagates along the coaxial transmission line until it reaches an impedance change. At this point, a portion of the signal is reflected back to the cable tester and is displayed as a change in amplitude. If the reflection point is lower in impedance than the cable, then the reflection will be displayed as a drop in amplitude. If it is higher in impedance, then it will be displayed as a rise in amplitude. The cable tester measures the time for a pulse to travel the distance between the beginning and end points of the probe, as displayed on the screen, and converts this time to a distance. Figure 2 shows a typical TDR trace with the probe connected to the instrument and inserted into a wet soil sample. It should be noted that the impedance of the probe assembly in the wet soil is lower than the cable, hence the amplitude of the return signal is lower. At the end of the probe assembly the impedance again changes (impedance increases) and is reflected in Figure 2 as a gradual rise in amplitude.

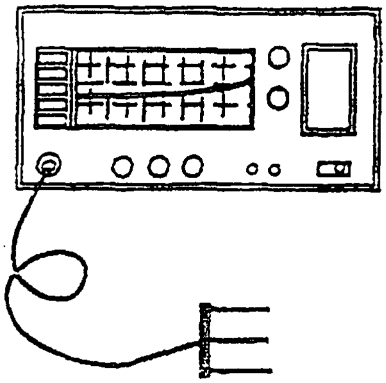


Figure 1. Basic TDR System

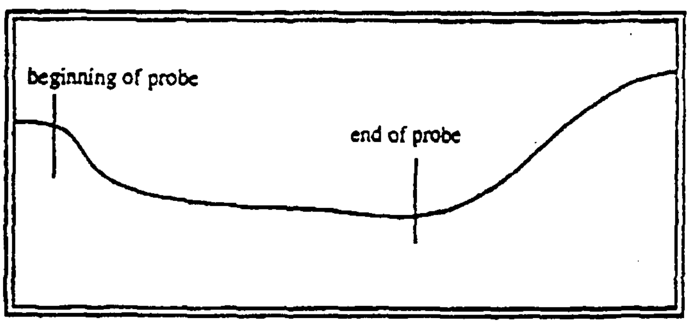


Figure 2. Typical TDR trace when the TDR probe is inserted into moist soil

7.2 TDR probes are typically divided into two categories: Two rod probes employing a balancing balun transformer, and multi-rod probes which do not require a balancing balun tranformer. Figure 3 is an example of a multi-rod TDR probe while Figure 4 is a typical two rod/balun TDR probe. Typically rod lengths, materials, diameters, and rod spacings vary from probe to probe. These parameters are chosen as a function of the soil to be tested, the longevity of the test, and the sensitivities required from the probe.

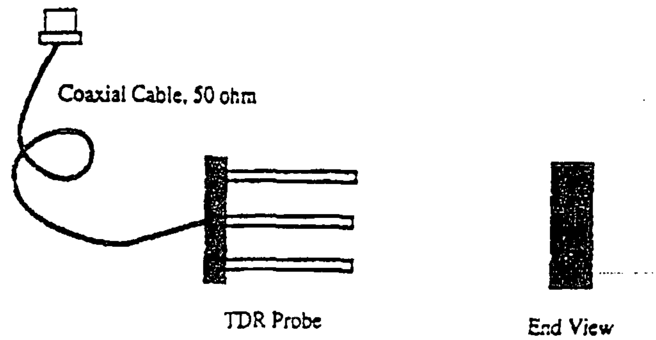


Figure 3. Typical 3-Rod TDR Probe Configuration

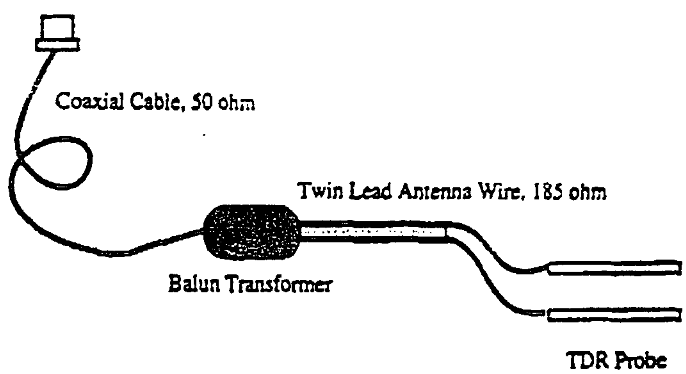


Figure 4. Typical 2-Rod TDR Probe With Balun Transformer

7.2.1 The two-rod balun probe makes use of a signal balancing balun. A Balun (also known as a balancing transformer) allows the user to connect two wires of dissimilar impedances. A typical example of impedance mismatch applicable to TDR probes is the 50 ohm coaxial cable connected to twin lead 185 ohm antenna wire. The balun transformer[b] is inserted at this junction so as to balance the impedance mismatch.

7.2.2 The balun transformer used in two rod probes has typically been a source of signal loss. A new type of balun has been developed [4] which alleviates the problems encountered with the typical balun.

7.2.3 Multi-rod, or multi-waveguide, probes employ, at a minimum, three conductive rods arranged in a symmetric pattern. The diameter of the rods, length, rod material, and spacing may vary.

7.2.4 Some commercially available systems offer the user multi-probe configurations, direct digital read out, data storage capabilities, probe lengths up to 120 centimeters long, time tagging of stored data, and real time data acquisition and control by computer through an RS232 serial interface.

7.3 With the use of a computer, and a series of multiplexers, a large number of TDR probes may be queried in a sequential fashion, and the data stored for later retrieval and analysis.

8.0 Preparation of Apparatus

8.1 Determining the proper propagation velocity (V_p) to be used in conjunction with the TDR probe is an important item in setting up a TDR probe system. The first step is to accurately measure the length of the cable and probe assembly. The propagation velocity may then be determined by performing the following procedure.

8.1.1 After attaching the TDR probe cable to the TDR cable tester, adjust the distance/division control to the appropriate setting. For example, if the cable/probe assembly is one meter, adjust the distance/division control to 1 m/div.

8.1.2 Turn the position adjustment until the distance reading is the same as the cable/probe assembly length.

8.1.3. Turn the propagation control (V_p) until the cursor is resting on the first rising portion of the reflected pulse, i.e., the end of the probe. Shorting the ends of the probe together will aid in determining the end of the probe as reflected in the TDR trace. The V_p controls of the instrument are now set to the V_p of the cable/probe assembly.

9.0 Calibration and Standardization

9.1 For best results, the TDR system should be calibrated to the soil to be tested. This can be accomplished by acquiring a sufficient quantity of the soil to be tested, so as to provide a minimum of seven soil samples, mixed to a uniform volumetric water content, as shown in Table 1.

9.2 Use an oven-drying or microwave-drying technique to remove any residual water from the soil samples. Periodic weighing of the sample is required to establish when the soil is dry. Two successive measurements of equal weight should be sufficient.

9.3 Using the dry soil, prepare seven soil samples having the volumetric water content as outlined in Table 1. Place the calibration samples into a plastic container. Do not use a metal container for the calibration samples. The calibration container should be of a sufficient size such that the radius of influence of the TDR probe is not affected by the container or surrounding air.

Table 1. Example of Volumetric Moisture Content of Calibration Samples

5%	10%	15%	20%	25%	30%	35%

9.4 Insert the probe into the calibration sample and, using the procedure outlined in Section 11. of this standard, calculate the volumetric water content. Repeat this procedure until all samples have been sampled at least three times.

9.5 Use an oven-drying or microwave-drying technique to quantify the actual water content of the soil samples used in the TDR calibration (as outlined in ASTM Standard D2216 or D4643).

9.6 If inaccuracies are noted then conduct a regression analysis. Inaccuracies are defined as repeated errors greater than plus or minus 2% (e.g., the calibration sample is mixed to a 15% volumetric water content and the user continuously arrives at a volumetric water content of 13% or less, or 17% or greater). The number of samples and the volumetric water percentages listed in Table 1 have been chosen so as to allow the user to conduct a representative regression analysis. A regression analysis will yield a mathematical relationship between the laboratory sample analyses of water content and the theoretical TDR calculations of water content (from equation 2). Should it be necessary to work with soils having a significantly large volumetric water content (>35%), then the user is encouraged to conduct a non-linear regression analysis.

9.7 No commercial calibration standards presently exist by which to calibrate the TDR probe.

10.0 Procedure.

10.1 Set the propagation velocity (v_p) as outlined in Section 8 of this test procedure.

10.2 With the probe in air, record the beginning and end points of the probe as indicated by the TDR instrument.

10.3 If small soil samples are to be tested, ensure that the containers used are large enough to allow insertion of the probe without touching either the bottom or sides of the container.

10.4 Insert the probe into the soil sample, taking care to ensure that no air gaps exist between the probe rods or the interface between the soil and probe base. Figure 5a depicts a TDR probe properly inserted into a soil sample while Figure 5b depicts a TDR probe improperly inserted into a soil sample. Air gaps decrease the accuracies observed. In addition, care should be taken to ensure that the waveguides remain parallel. If the waveguides are not parallel, inaccuracies will result. Record the beginning and end points of the probe while inserted in the soil.

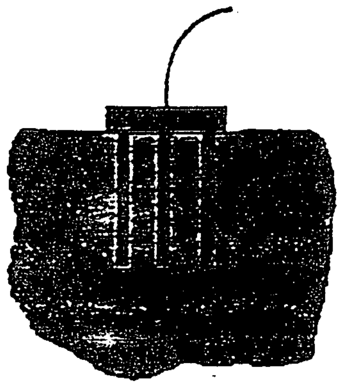


Fig. 5a - Properly installed probe

Fig. 5b - Improperly installed probe

Figure 5. Properly and Improperly Installed TDR Probes.

10.5 The beginning point should be the same as noted with the probe in air. If the beginning point is not the same, remove the probe from the calibration sample, and check for damage. If the probe is found to be in proper working order, reinsert the probe into the calibration sample, and again note the beginning and end points.

10.6 If the beginning point is the same as that found in air then continue with the sampling; if not, cease sampling, and check the system for damage or improper instrument settings. Repair or readjust as required.

10.7 Insert the probe into the sample following instructions 10.3 through 10.6. Determine the apparent trace length, l_a , as shown on the TDR instrument. It is very important to be able to properly determine the correct beginning and end points of the probe when inserted into a soil sample. The end point, as observed on the TDR instrument, will vary with water content. Improperly determining the trace end point will introduce gross errors in the predicted water content. Using the equations outlined in Section 11, calculate the volumetric water content.

10.8 Leave the probe in the sample and repeat the measurement and the calculation. The soil should be sampled a minimum of three times to ensure repeatability and accuracy. The same sampling rate is applied to soils sampled in the field.

10.9 For systems which have been set-up for long term monitoring, a periodic operational check of the system should be conducted.

11. Calculation

11.1 Time Domain Reflectometry (TDR) is a method which can be used to measure the volumetric water content of soil, θ_v (where $\theta_v = V_w/V_t$, V_w = volume of water [L^3], and V_t = total volume of soil [L^3]). A unique relationship exists between the volumetric water content of soil (θ_v) and its dielectric constant, K_a, where $\theta_v = f(K_a)$ [5] .

11.1.1 The dielectric constant, K_a, is related to the apparent length (l_a) of the probe as shown in equation 1.

$$K_a = [l_a/(l_p*v_p)]^2 \qquad (1)$$

where:

l_p = actual probe rod length [L]
l_a = apparent length of the probe rod as determined from the TDR trace [L]
v_p = propagation velocity of the signal [L/T]

l_a is determined by subtracting the beginning point of the probe, l_{begin}, from the end point of the probe, l_{end}, while the probe is inserted into the soil sample. Once the apparent length of the probe, l_a, is calculated, this information is used to determine the dielectric constant (K_a) of the soil (Equation 1). Topp [5] determined that the dielectric constant of soil is related to the volumetric moisture content by the empirical relationship:

$$\theta_v = -0.053 + (0.0292)K_a - (5.5x10^{-4})K_a{}^2 + (4.3x10^{-6})K_a{}^3 \qquad (2)$$

In reality, the relationship between actual and predicted moisture content has some uncertainty due to differences in the dielectric of the media. Therefore a regression analysis is typically performed to develop a calibration relationship for better accuracy between observed versus TDR results.

12. Report

12.1. Report the volumetric water content to the first decimal (tenths of a percent) when the average of replicate values permits.

12.2. Report the repeatability of the measurements. Accuracies should not vary by more than ±2% volumetric water content.

12.3 Report the mean, standard deviation, and the standard error.

12.4 Report the calibration equation used and the method used to determine the equations coefficients.

12.5 Report the date, time, and location where the sampling occurred.

12.6 Report the probe configuration, i.e., two-rod with balun, three rod, rod length, etc.

13. Precision and Bias

13.1 Bias

13.1.1 The resolution of the TDR measurement depends on the resolution of the TDR instrument, the properties of the soil (electrical properties and dielectric constant), cable length (long cables attenuate the amplitude of the return signal and contribute to noise (e.g., antenna effects)), probe configuration, technique used to analyze the TDR data, and operator skills and experience.

13.1.2 A calibration procedure, as outlined in section 9, is recommended for optimum performance of a TDR system. A regression analysis of laboratory moisture content values versus theoretical TDR calculated moisture content (as defined by equation 2) should yield a relationship that will estimate moisture contents within ±1% by volume.

13.2 Precision

13.2.1 Precision of this test method is established by statistical analysis of repeated measurements of a TDR system. Holding all experimental conditions constant, thirty (30) TDR measurements of moisture content are made on a soil sample within a range of interest for moisture. The mean and standard deviation are calculated from these data. Experience has shown that experiments holding all variables constant have resulted in measurement errors of ± 2% for θ_V. If variations larger than ± 2% are encountered for multiple runs using standards, results should be considered suspect, and the tests should be repeated. If the data are calibrated to actual soil analyses through regression, then ±1% precision may be expected [7].

13.3 Multilaboratory precision.

13.3.1 There is no multilaboratory precision information at this time.

14. Keywords

14.1 Time Domain Reflectometry (TDR)
Volumetric Water (Moisture) Content
TDR Probes
Balun Transformer
Propagation Velocity
Multiplexers
Soil samples

15. References

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[7] Zegelin, S.J., White, I, and Russell, G.F., "A Critique of the Time Domain Reflectometry Technique for Determining Field Soil-Water Content", Advances in Measurement of Soil Physical Properties: Bringing Theory into Practice. Soil Science Society of America, Special Publication no. 30, pp. 187-208, 1992.

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[a] The Tektronix 1502B is the instrument around which the TDR probe technology has been developed. With rare exception, commercial companies selling a TDR probe system employ this instrument in their systems.

[b] A typical balun used here is the Type T.P. 103 Balun as sold by the Adams - Russel Co., of Burlington, MA.

APPENDIX K

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FRACTURE WELL MODELING STATUS/METHODOLOGY

FRACTURE WELL MODELING STATUS/METHODOLOGY

REMEDIATION OF ZONE 1
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT

November 1995

Prepared by:	Frederick A. Mueller, P.E. Project Environmental Engineer	11/7/95 Date
	Robert D. Bowden, Jr. Project Hydrogeologist	11/7/95 Date
Reviewed by:	David L. Bramley, P.E. Project Manager	11/7/95 Date
Project Director:	Christopher R. Klemmer, P.E. Vice President	11/7/95 Date

FRACTURE WELL MODELING STATUS/METHODOLOGY

REMEDIATION OF ZONE 1
LINEMASTER SWITCH CORPORATION
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1.0 INTRODUCTION

1.1 Background

The Record of Decision (ROD) for the Linemaster Switch Corporation site requires dewatering of the Zone 1 overburden and application of soil vapor extraction to remove VOCs from the Zone 1 soils and groundwater. As presented in the Conceptual Design Report (Fuss & O'Neill Inc. (F&O), 1995), to remediate the Zone 1 area as delineated in the Draft Remedial Investigation/Feasibility Study (RI/FS) (Figure 1.1), a dewatering/soil vapor extraction (SVE) system is proposed. Dewatering/SVE is a remedial method whereby both groundwater and soil vapors are extracted from the same recovery well. The extraction of groundwater dewaters the formation so that soil vapor can be extracted from the formation under a vacuum.

A "soil fracturing pilot test" is being conducted to assess the feasibility of hydraulic soil fracturing to enhance groundwater extraction for dewatering and increase subsurface vapor flow and provide dewatering/SVE system design criteria. The proposed plan for the soil fracturing pilot test is presented in the "Draft Work Plan for Pilot Test to Evaluate the Feasibility and Effects of Hydraulic Soil Fracturing on Fluid Recovery, Linemaster Switch Corporation, Woodstock, Connecticut" (F&O and FRX, 1995).

1.2 Modeling Objectives

The modeling effort detailed in this report is intended to be employed in conjunction with the soil fracturing pilot test results to provide a means for evaluating the pilot test results, evaluating the feasibility of implementing fracturing at the site and, if appropriate, aiding in the design of a full scale remedial system. Detailed three-dimensional modeling is necessary to evaluate the subsurface air and groundwater recovery rates in unsaturated and saturated overburden deposits due to the complex geometry of hydraulically fractured recovery wells. Specifically, the objectives of the fracture well modeling are to:

Pre-Pilot Test Modeling Objectives

- Estimate the time to achieve a steady-state subsurface vacuum distribution;
- Assess the sensitivity of the time to reach a steady-state subsurface vacuum distribution to the effective air porosity of the soil;
- Predict the steady-state subsurface vacuum distribution;
- Provide an estimate of the air recovery rate from a fracture well;
- Predict the lateral and vertical extent of overburden dewatering that may occur during field testing; and
- Provide an estimate of the short-term groundwater recovery rate from a fracture well.

Post-Pilot Test Modeling Objectives

- Update the current estimates of the site saturated zone and vadose zone parameters by using the data collected during the dewatering/SVE recovery test portion of the soil fracturing pilot test to calibrate the air and groundwater flow models.

- Make projections of long-term dewatering/SVE recovery well performance to aid in the design of a dewatering/SVE recovery wellfield, and to determine if a full-scale dewatering/SVE remedial system using fracture wells is feasible.

2.0 ZONE 1 GEOLOGY/HYDROGEOLOGY

The geology of the Zone 1 area at the Linemaster site consists of unconsolidated glacial till deposits overlying schist bedrock. The overburden is the geologic unit of concern for the dewatering/SVE remediation. The Draft RI (F&O, 1992) determined that the site overburden deposits are glacial till sediments consisting of a dense, compact, non-sorted and non-stratified mixture of clay, silt, sand, gravel, cobbles, boulders, and angular rock fragments. In the Zone 1 area, the thickness of the till ranges from approximately 40 to 50 feet, and increases to the north. The depth to groundwater of the till layer in the Zone 1 area ranges from 3 to 15 feet below grade (ftbg). At the eastern edge of Zone 1, where the soil fracturing pilot test will be conducted, the till ranges in thickness from 40 to 55 feet, and the depth to groundwater is estimated to range from approximately 12 to 15 ftbg.

2.1 Hydraulic Conductivity

Hydraulic conductivity is a measure of the capacity of a porous medium to transmit water, and is a function of both the medium (size and shape of pores and degree of interconnection between pores) and the physical properties of the fluid flowing through it (i.e., viscosity and density) (Driscoll, 1986). The hydraulic conductivity is an important hydraulic parameter; the total flow (Q) through any cross-sectional area of an aquifer (A) can be calculated using the hydraulic conductivity (K) and the horizontal hydraulic gradient (i), using one form of the Darcy equation, $Q = KiA$. The transmissivity of an aquifer is the product of the hydraulic conductivity and the aquifer thickness, and describes the rate at which water is transmitted through a unit width of the aquifer.

As summarized in Attachment A of the Conceptual Design Report, hydraulic conductivity estimates of the Zone 1 till were determined using three methods: slug test data analyses, pumping test data analysis, and grain size analysis by the Hazen Method. The hydraulic conductivity of the till in the Zone 1 area is estimated to range from approximately 0.01 to 0.001 feet per day (ft/d) (3.53×10^{-6} to 3.53×10^{-7} centimeters per second (cm/s)). The greater value is likely representative of the shallow till (less than approximately 15 to 20 ftbg) and the lower value likely represents the deeper till (greater than approximately 15 to 20 ftbg). A more detailed description of the overburden characteristics is included in the Conceptual Design Report.

2.2 Storativity

The storativity, or storage coefficient, of an aquifer can be defined as the volume of water that the aquifer releases from or takes into storage per unit surface area of aquifer per unit change in the hydraulic head (Freeze and Cherry, 1979); storativity is dimensionless. An aquifer's storativity, along with the transmissivity, governs the amount of water the aquifer can yield in response to stresses such as pumping. It is necessary to specify the storativity when performing a transient (time-dependent) simulation because water is released from or taken into storage within the aquifer during the transient response of an aquifer to an induced stress. When the transfer to and from storage stops, the system reaches steady state and the heads stabilize (Anderson and Woessner, 1992).

As detailed in Appendix J of the Draft RI, a 25-day pilot pumping test conducted during March and April 1992 at the overburden dewatering well DW-1t, located just east of the main facility in the middle of Zone 1, yielded a mean till storativity of 0.004 ($4x10^{-3}$). The dense, compact and non-sorted till, however, has a very low capacity to transmit and yield water. The pilot pumping test yielded an average long-term overburden pumping rate of only 0.03 gallon per minute (gpm). The pump test results indicated that the saturated overburden at the Linemaster site exhibits the characteristics of an unconfined aquifer, where water is released by dewatering the soil pores. However, the low overburden storativity value may indicate that the extraction of groundwater from the saturated overburden storage may reflect some degree of semi-confined aquifer response, where some portion of the water is released due to aquifer compression and water expansion caused by changes in the fluid pressure.

2.3 Effective Air Permeability

Effective air permeability is a measure of the capacity of a porous medium to transmit air and is a function of the intrinsic permeability of the soil and the moisture content. Effective air permeability can be estimated by multiplying the intrinsic permeability of the soil by the relative permeability. The intrinsic permeability is a function of the soil matrix determined directly from the hydraulic conductivity. The relative permeability, k_r, varies from zero to one and describes the variation in air permeability as a function of air saturation (or moisture content). As soil moisture increases, the amount of pore air space decreases. Limited data exist on relative permeability. For a clay matrix, EPA (1992) reported a relative permeability of 0.1 at a soil moisture of approximately 35 percent by volume. It is likely that the soil matrix at Linemaster demonstrates similar characteristics. Consequently, a conservative value of 0.1 was selected for relative permeability of the till.

Estimates of Zone 1 overburden permeability ranges are as follows:

Intrinsic Permeability, $k_i = 3.60x10^{-11}$ to $3.60x10^{-12}$ cm^2

Relative Permeability, $k_r = 0.1$

Effective Air Permeability, $k_a = 3.60x10^{-12}$ to $3.60x10^{-13}$ cm^2

The higher values are representative of the shallow till and the lower values are representative of the deeper till. The intrinsic permeability given above falls within the reported range for glacial till (Freeze and Cherry, 1979).

2.4 Effective Air Porosity

The total porosity of a soil matrix is the ratio of total void volume to the total unit volume of the soil; porosity is dimensionless. The void volume can be occupied by air, water, or a combination of both. For a dried soil sample, porosity values for silts range from 0.35 to 0.50 (Freeze and Cherry, 1979).

The effective air porosity of a soil is the ratio of the pore volume of connected air spaces to the total unit volume of the soil. The effective air porosity is dependent on the moisture content of the soil. The effective air porosity must be lower than the total porosity. It is necessary to specify an effective air porosity when performing transient simulations because of the compressibility of air in the formation. It is also necessary to understand the average velocity of air molecules through the till. For the air flow modeling, effective porosity values of 0.1 and 0.01 were used for the transient simulations to cover the anticipated range of the actual effective air porosity of the upper till.

3.0 FRACTURE WELL AND PILOT TEST DESIGN

The soil fracturing pilot test will consist of the installation of two fracture wells and a dewatering/SVE recovery test in which submersible pumps and vacuum blowers will be used to extract groundwater and soil vapors from the two fracture wells. The pilot test area, located in the eastern portion of Zone 1, will have strategically placed multilevel piezometers and a water-impermeable cap.

3.1 Fracturing Technique and Pilot Test Layout

Hydraulic fracturing involves the injection of sand-laden fracturing fluid under pressure into the subsurface to create fractures to enhance the advective transport of vapor and/or water. The hydraulic fracturing methodology to be employed for the two fracture wells is summarized in the pilot test Draft Work Plan. The fracturing fluid decomposes soon after injection, leaving the sand to prop the fractures open.

The two wells from which the fracturing will be initiated, FW-A and FW-B, have been installed 70 feet apart in the eastern section of Zone 1 as shown in Figure 3.1 (the fracture well designations have been reversed since their installation). A monitoring network of ten multi-level piezometer clusters will be installed adjacent to the fracture wells to monitor hydraulic head and pressure distribution in the subsurface. Some of the piezometer clusters will be installed after the creation of the soil fractures. Soil samples will be collected during the installation of these piezometer clusters to identify fracture locations. A series of tests will then be conducted using the fracture wells and the monitoring network to evaluate the groundwater potentiometric response, subsurface vacuum distribution, and vapor and fluid recovery during the dewatering/SVE recovery test portion of the soil fracturing pilot test. Figure 3.2 provides plan and cross-section views of the fracture wells, the piezometer clusters to be used during the soil fracturing pilot test.

3.2 Fracture Well Design

3.2.1 Fracture Depths, Thicknesses and Radii

The original soil fracturing pilot test plan, "Proposal, Pilot Scale Test to Evaluate the Feasibility and Effects of Hydraulic Fracturing on Fluid Recovery" (FRX, 1995) specified two eight-fracture pilot test wells. Based on subsequent modifications and discussions concerning the effects of heaving under the Linemaster facility building, the original plan underwent several revisions prior to arriving at a layout consisting of one seven-fracture well and one three-fracture well.

The depth to bedrock at each soil fracture well location was assumed to be 40 ftbg. The eight-fracture well fractures were proposed to be spaced every four feet, with the top fracture located at eight ftbg and the bottom fracture lying at 38 ftbg, two feet above bedrock. The three-fracture well fractures were proposed to be spaced every 15 feet, with the top fracture located at eight ftbg and the bottom fracture lying at 38 ftbg, two feet above bedrock.

Based on subsequent design modifications incorporating additional geologic data (including updated depths to bedrock) and dewatering/SVE system design limitations, a revised soil fracturing scheme was devised. Four hydraulic fractures would be created from FW-A and eight fractures would be created from FW-B. The FW-A fractures generally would be created with a ten-foot vertical spacing, at depths of 8, 18, 28 and 33 ftbg. The FW-B fractures would be created with a five-foot vertical spacing, at depths of 8, 13, 18, 23, 28, 33, 38 and 43 ftbg. The fractures would therefore be located from approximately 7 to 49 feet above the bedrock surface, which lies at depths ranging from approximately 40 to 56 ftbg in the pilot test area. Under ideal conditions, fractures generally will propagate horizontally and result in a circular pattern in plan view. Typically, however, fractures may rise as they extend from wells and result in an elliptical pattern. The anticipated maximum propped thickness of each fracture was approximately 0.8 to 1.0 cm (0.026 to 0.033 ft) (F&O and FRX, 1995).

The lateral extent of fracture propagation typically increases with depth. Theoretical calculations indicate that the fractures will range in propped radius from approximately 12 to 34 feet. As shown in Figure 3.2, the resulting zone of stacked fractures will have an inverted cone-shaped perimeter.

3.2.2 Proppant Sand Characteristics

Based on information provided by FRX, it is anticipated that the sand-filled fractures will have the hydraulic conductivity of 150 ft/d (5.29×10^{-2} cm/s). Based on this value, the sand "proppant" intrinsic permeability would be 5.39×10^{-7} cm^2. Since the proppant sand material is a well-sorted (uniform) sand, its ability to retain moisture is much less than that of till. Consequently, a relative permeability value of 0.5 was selected for the sand-propped fractures. The effective air permeability of the fractures was then calculated to be 2.70×10^{-7} cm^2. The effective air porosity of the fractures was assumed to be 0.3, since the uniform proppant sand should drain freely. A storativity of 0.01 (1×10^{-2}) was selected for the proppant sand (Freeze and Cherry, 1979).

3.2.3 Control of Air Flow and Groundwater Extraction

Each fracture well will be equipped with a submersible pump capable of extracting groundwater in a controlled manner from all fractures. A vacuum blower will be connected to each well so that the soil vapors can be extracted from the well under vacuum. The wells will be constructed in a manner that allows vacuums to be placed either on all fractures or on alternating fractures. The fractures not placed under a vacuum will be exposed to atmospheric pressure to allow passive air injection.

3.3 Area Cap

A water-impermeable cap will be placed over the exposed Zone 1 areas to minimize vertical infiltration of precipitation during the test. Runoff will be directed away from the facility toward natural slopes and swales. This cap will not have a sufficiently low air permeability to constitute a vapor cap.

3.4 Evaluation of Fracture Well Performance

A series of tests will be conducted using the fracture wells and the monitoring network to evaluate fluid and vapor recovery rates during the dewatering/SVE recovery test portion of the soil fracturing pilot test. The fracture wells will be operated in multiple configurations during the dewatering/SVE recovery test. The first phase of the recovery test will involve dewatering of the overburden by extracting water and vapor (under applied vacuums ranging from 10 to 14 inches Hg) from all fractures in both wells. During the second phase, dewatering will continue while the pressures applied to alternating fractures of the two wells are manipulated between atmospheric pressure and vacuums of 10 to 14 inches Hg.

During the first phase, it is expected that only the top two fractures will be dewatered; the air flow model was therefore designed to simulate only the vertical interval containing the top two fractures. The groundwater potentiometric level and extraction rate data collected during the first phase will be used to calibrate the fracture well groundwater flow model. The soil vacuum distribution and vapor extraction rate data collected will be used to calibrate the air flow model. The soil vacuum distribution and vapor extraction rate data collected during the second phase will be used to refine the air flow model calibration.

4.0 AIR AND GROUNDWATER FLOW MODELS - COMMON ELEMENTS

The finite-difference air flow model AIR3D (Geraghty & Miller, Inc., 1994) was used to simulate the air flow in the unsaturated soil. The finite-difference three-dimensional groundwater flow model MODFLOW (McDonald and Harbaugh, 1988) was used to simulate the response of the saturated overburden to dewatering activities.

To provide a numerical approximation of the partial differential equation which describes the three-dimensional movement of air or groundwater through porous earth material, AIR3D and MODFLOW employ the finite difference method. A continuous flow system is spatially discretized into a finite three-dimensional mesh of points discrete in space and time, and the partial derivatives are replaced by terms calculated from the differences in head values at these points. The process leads to systems of simultaneous linear algebraic difference equations; their solution yields values of head at specific points and times. These values constitute an approximation of the time-varying head distribution that would be given by an analytical solution of the partial-differential equation of flow (McDonald and Harbaugh, 1988).

4.1 Modeling Approach

The three-dimensional air and groundwater flow models were set up with similar boundary conditions, grid configurations and layer spacings surrounding fracture-containing layers. The intent in matching the model layouts was to minimize differences between the air and groundwater flow models attributable to variations in the model framework. The coordination of the two models was facilitated by the fact that the AIR3D air flow model is an adaptation of the MODFLOW groundwater flow model, and uses MODFLOW for its computational platform.

The MODFLOW program structure consists of a main program and a series of highly independent subroutines called "modules". The modules are grouped into "packages". Each package deals with a specific feature of the hydrologic system which is to be simulated or with a specific method of solving linear equations which describe the flow system, such as the strongly implicit procedure or the preconditioned conjugate-gradient method (McDonald and Harbaugh, 1988).

4.2 Model Design and Setup

The air and groundwater flow models focused primarily on simulating the flow surrounding and into a single fracture well in response to vapor extraction and dewatering. An additional groundwater simulation superimposed the effects of two fracture wells, as discussed in Section 6.3.3; this was intended to provide a predictive simulation of the first phase of the dewatering/SVE recovery test portion of the soil fracturing pilot test.

4.2.1 Boundary Conditions and Symmetry

In order to reduce the model dimensions and input and output files to a manageable size, the fracture well models were configured to simulate one plan view quadrant of the overall area to be modeled. This was accomplished by assuming radial symmetry within the flow field surrounding a modeled well. The grid layout for the models is shown in Figure 4.1. The simulated location of the well borehole was set at the lower left plan-view corner of the model, and no-flow boundaries were positioned along the two model edges adjacent to the well. The resulting model is therefore symmetrical across the no-flow boundary axes; the symmetry is used to represent the fracture well configuration in all four plan view quadrants.

The extent of the modeled space was selected to be large enough so that the model results would be insensitive to the boundary conditions at the edge of the model. A distance of 30 meters (approximately 100 feet) was used in the horizontal dimensions of the model. Constant pressure (air flow model) or constant head (groundwater flow model) boundaries were established along the two model edges opposite the well. A constant pressure or constant head boundary fixes the pressure or potentiometric head in specified cells and allows the adjacent boundary flux to vary until the governing cell-by-cell flow equations are satisfied.

Both the air and groundwater flow models require a no-flow boundary at the bottom of the model. This boundary condition is an approximate representation of the water table in the air flow model and of the bedrock surface in the groundwater flow model. In addition, both models require a specified-head boundary at the top of the model. To simulate this boundary condition, the AIR3D model establishes an "invisible" layer of constant atmospheric pressure head positioned above the top model layer. In the groundwater flow model, the active top-layer cells were designated variable-head boundaries, and were assigned initial hydraulic head values representing the static water table.

4.2.2 Grid Configuration

4.2.2.1 Model Representation

AIR3D and MODFLOW utilize the "block-centered" finite difference method, where the flow regime is discretized into three-dimensional blocks or cells. Each cell is centered by a point called a "node" for which the head is to be calculated. The node represents only the average spatial point for the cell; likewise, the block-centered finite difference model solves for the average head in each cell, not for the exact value at any one point. It follows that with decreasing (tighter) grid spacing, the precision of the solution improves, particularly adjacent to source or sink (withdrawal) stress points (e.g., vapor and/or groundwater recovery wells), which can effect large differences in head over short distances.

With increasing distance from a stress point, the grid spacing will have progressively less influence upon the solution precision. Progressively increasing the nodal spacing outward from stress points toward the boundaries is practical in that it provides coverage of the modeled area while minimizing data handling and computer storage and computation time. In order to maintain numerical stability during the numerical solution process, the difference in spacing between two adjacent nodes must be by a factor of less than 1.5. The air and groundwater flow models utilized variable grid spacing; this was done to maximize the solution precision adjacent to the stress points (fracture locations), while expanding cell dimensions with increasing distance outward from the fractures to avoid generating unnecessarily large data and output files.

4.2.2.2 Horizontal Grid

As shown in Figure 4.1, in the X and Y directions the air and groundwater flow model grids were designed using variable nodal spacing, with the smallest cells concentrated at the borehole (i.e., lower left) corner in plan view. The total distance in the X and Y directions was set at 3,000 cm (98.43 ft), and the number of rows and columns was set at 30 each. Based on a minimum row/column spacing of 7 cm (0.23 ft), which is consistent with the dimensions of one quadrant of the fracture well borehole, the scaling factor was calculated to be 1.1493. The grid spacing thus increased outward from 7 cm (0.23 ft) at Row 30, Column 1 to 395.8 cm (12.98 ft) at Row 1, Column 30. As discussed in Section 4.2.1, no-flow boundaries were positioned along Row 30 and Column 1, and constant-head boundaries were established along Row 1 and Column 30.

4.2.2.3 Vertical Grid

In the vertical (Z) direction, the model grids also incorporated variable layer spacing in representing the horizontal fractures and the overburden intervals between them. Figure 4.2 is a section view of a fracture well showing the fractures and the corresponding model layers. Due to a limited time frame available for constructing and refining the model prior to the initiation of the dewatering/SVE recovery portion of the fracture well pilot test, it was necessary to begin constructing the model using the number, depths and spacing of fractures specified in one of the fracturing plan revisions prior to reaching the final fracture plan. The fracturing scenario modeled specified one eight-fracture well (with seven fractures within the saturated zone) and one three-fracture well (with two fractures within the saturated zone). The depth to bedrock in the fracture well locations was modeled at 40 ftbg. For both models, the eight-fracture well fracture spacing was four feet, with the top fracture located at eight ftbg and the bottom fracture lying at 38 ftbg, two feet above bedrock. For the groundwater model, the three-fracture well fracture spacing was 20 feet, with the top saturated-zone fracture located at 18 ftbg and the bottom fracture lying at 38 ftbg, two feet above bedrock.

Six model layers were inserted between each of the fractures to maintain a manageable model size. In order to maintain a thickness scaling factor of less than 1.5 between each layer, a minimum layer thickness of 10 cm (0.33 ft) was selected. The resulting scaling factor was calculated to be 1.3465. Therefore, as illustrated in Figure 4.2, the thickness of each model layer containing a fracture was 10 cm (0.33 ft) and the thicknesses of the inter-fracture layers progressively increased to a maximum thickness of 24.4 cm (0.80 ft) for the "mirrored" pair of layers located midway between each fracture.

4.2.3 Fracture Representation

For the model layers containing fractures, the estimated areal extent of the fracture in plan view (X-Y) was delineated. Figure 4.3 depicts the plan-view segregation of a typical fracture-containing layer into the fractured and non-fractured regions. All cells falling within the fracture perimeter were assigned fracture-specific equivalent pneumatic or hydraulic parameter values.

The model layers containing fractures are 10 cm thick. However, the actual propped thicknesses of the fractures are expected to be 0.8 to 1.0 cm. Because the 10 cm (0.33 ft) thickness of the model layers containing the fractures is much thicker than the expected thickness of the high-permeability sand-propped fractures, equivalent horizontal and vertical effective air permeabilities and hydraulic conductivities were calculated for the fracture zones within each of the fracture layers using the following equations (Anderson and Woessner, 1992):

$$K_x = \sum_{k=1}^{2} \frac{K_k b_k}{B} \tag{1}$$

$$K_z = \frac{B}{\sum_{k=1}^{2} b_k / K_k} \tag{2}$$

where K_x = the equivalent horizontal effective air permeability (air flow) or hydraulic conductivity (groundwater flow) of a layer containing a fracture;
K_z = the equivalent vertical effective air permeability or hydraulic conductivity of a layer containing a fracture;
K_k = the effective air permeability or hydraulic conductivity of the till (k=2) or fracture proppant sand (k=1);
b_k = the propped thickness of the fracture (k=1) (0.8 to 1.0 cm (0.026 to 0.033 ft)) or of the till (k=2) in the model layer; and
B = the thickness of the model layer (10 cm or 0.33 ft). Note that $b_1 + b_2 = B$.

Using equations (1) and (2), the following equivalent horizontal and vertical hydraulic conductivities and effective air permeabilities were calculated for the modeled fracture zones within the fracture layers:

Equivalent Hydraulic Conductivities:

Upper till: K_x = 15.009 ft/d (5.29×10^{-3} cm/s)
K_z = 0.011 ft/d (3.92×10^{-6} cm/s)
Lower till: K_x = 15.001 ft/d (5.29×10^{-3} cm/s)
K_z = 0.001 ft/d (3.92×10^{-7} cm/s)

Equivalent Effective Air Permeabilities:

Upper till: $k_x = 2.78 \times 10^{-8}$ cm^2
$k_z = 3.99 \times 10^{-12}$ cm^2

The hydraulic conductivities, storativities, intrinsic permeabilities, effective air permeabilities and effective air porosities used for the modeled upper till, lower till, fractures and fracture layers are summarized in Table 4.1.

4.3. Results Interpretation

Since the modeled fracture well is based on symmetrical radial flow to the well, the results of the model can be interpreted by analyzing the results along a representative cross-section of the model. Row 29, which intersects the cell where the fracture well is located, was used in both models as this representative cross-section.

5.0 AIR FLOW MODEL

5.1 Model-Specific Input Parameters

A 17-layer air flow model was constructed to represent the two upper fractures within the unsaturated zone for the eight-fracture well. The two saturated zone fractures were simulated at depths of 8 and 12 ftbg (in Layers 8 and 15 of the model). While the initial water table at fracture well is projected to be at a depth of 10 ftbg, the air flow model is based on a water table that is 14 ftbg. This depth was used to account for dewatering that was anticipated to occur during the test and to allow for the evaluation of air flow to and between the top two fractures.

Since only the top 14 feet of soil was simulated in the air model, the parameters listed in Table 4.1 for the upper till and upper till sand-propped fractures were used. The temperature of the air/soil system was assumed to be 10°C. The air viscosity used in the model was 1.76×10^{-4} g/cm-sec. The effective air porosity values discussed in Section 2.4 were used in the transient simulations. As described in Section 4.2.1, constant atmospheric pressure (1.0 atmospheres) was assumed for the surface of the model and at a constant pressure boundaries along the two model edges opposite the well. The remainder of the model was set up as described in Section 4 except as where noted below.

5.2 Simulation of Fracture Well

The AIR3D model simulates a vapor extraction well as a constant pressure boundary at the well location in the grid. The grid cell representing one quadrant of the well, was set to a constant pressure of either 0.6 atmosphere (12" Hg vacuum) or 1.0 atmosphere, depending upon the fracture and desired flow configuration.

Based upon the specified pressures at the fracture well, the constant pressure boundary conditions at the top and edges of the model, the site geometry and site soil properties, the pressures in the model cells are calculated by AIR3D. Air flow within the soil and fracture, which is caused by pressure gradients, between the cells is also computed by AIR3D.

AIR3D allows the volumetric and mass air flow rates into or out of constant pressure boundary conditions (i.e the wells, the model surface, and the model edges) to be determined. This allows the air flow rate into or out of a particular fracture can be determined.

5.3 Steady-State Air Flow

Steady-state simulations were conducted to determine the vacuum distributions and flow rates resulting two different vacuum configurations. In Example 01, a vacuum of 0.6 atmosphere was applied to both fractures in the well. In Example 02, the top fracture pressure was set to 0.6 atmosphere and the bottom fracture pressure was set to 1.0 atmosphere. The resulting steady-state pressure contours are shown as Figure 5.1 and Figure 5.2. The volumetric flow rates for these two examples are given in Table 5.1.

The radius of influence, as measured by the 0.98 atmosphere contour line, is about 25 percent larger for the two-fracture vacuum test (Example 01), but the air flow rate into the soil is about 30 percent less than for the one-fracture vacuum test (Example 02). Therefore, the scenario where one fracture is under vacuum while the other is at atmospheric pressure will provide more intense treatment of the soil that is between and near the fractures. This intensity is seen in Figure 5.2 by the larger number of contour lines between the fractures. Since these contours are evenly spaced between the fractures, a uniform pressure gradient is expected in the pilot tests, in contrast to the two-vacuum configuration of Example 01 where little gradient is expected. The scenario where both fractures are under vacuum will provide more intense treatment of the soil that is beyond the edge of the fractures. This is evidence in Figure 5.1 by the larger number of contour lines in the area outside the fractures.

The results of both simulations indicate that there is a negligible pressure drop within the fractures. This is due to the air permeability of the fracture being several orders of magnitude greater than the soil. This model suggests that the fracture could be modeled as a horizontal layer of constant pressure (or vacuum) within the soil. In both models, there was a negligible amount of air flow from the edges of the model. This indicates that the model size was selected appropriately.

5.4 Transient Air Flow

Several transient variations of the two-vacuum case from Example 01 were made to estimate the time required for the subsurface vacuum distribution to reach steady state. The pressure distribution in the soil after one hour and after 24 hours for an effective air porosity of 0.1 are shown in Figures 5.3 and 5.4, respectively. Within one hour the subsurface pressure gradients have propagated primarily within the fractures and it is not until after 24 hours that the subsurface pressure gradients have approached steady state values.

The pressure distribution in the soil after one hour and after 3.3 hours for an effective air porosity of 0.01 are shown in Figures 5.5 and 5.6, respectively. Within one hour the subsurface pressure gradients have already propagated past the fractured area and after 3.3 hours the subsurface pressure gradients have approached steady state values.

6.0 GROUNDWATER FLOW MODEL

6.1 Model-Specific Boundaries and Input Parameters

A 51-layer groundwater model was constructed to represent both the seven (or two) fractures within the saturated zone and the surrounding saturated overburden intervals. For the modeled eight-fracture well, the seven saturated zone fractures were simulated within Layers 6, 13, 20, 27, 34, 41 and 48. For the modeled three-fracture well, the two saturated zone fractures were simulated within Layers 13 and 48. MODFLOW allows only Layer 1 to be modeled as "unconfined"; however, since the remaining overburden will be subject to dewatering, the remaining 50 layers were simulated as "confined/unconfined" layers. This MODFLOW option allows for variable transmissivity in a lower model layer should the potentiometric head drop to below the top of the layer. In addition, the storativity within a layer may alternate between confined and unconfined values in response to dewatering, and the vertical leakage into a layer from the layer above it will be limited if the layer desaturates (McDonald and Harbaugh, 1988).

The model leakage term is a function of the vertical leakance, which is the ratio of the vertical hydraulic conductivity of a layer to the thickness of the interval from the midpoint of that layer to the midpoint of the layer beneath it. For this flow model, the overburden was assumed to be isotropic within each layer; the MODFLOW pre-processor thus calculated the vertical leakance at each cell location from the input layer horizontal hydraulic conductivities and thicknesses.

As discussed in Sections 2.2 and 3.2.2, the model confined and unconfined storativity values were $4x10^{-3}$ and $1x10^{-2}$ for the overburden and fracture portions, respectively. The storativity was constant for each layer representing the overburden because dewatering of a layer was not expected to result in a higher "specific yield" storativity.

Because an area cap will be placed over the exposed Zone 1 areas to reduce atmospheric water infiltration during the dewatering/SVE recovery test portion of the soil fracturing pilot test, the effects of areal recharge (precipitation infiltration) and evapotranspiration were not included in the groundwater model.

The remainder of the model was set up as described in Section 4 except as where noted below.

6.2 Simulation of Fracture Well

The fracture wells represent sinks, or locations where water is withdrawn from the model. Unlike the AIR3D model, the MODFLOW model cannot simulate the establishment of a negative pressure head for the purpose of extracting water from the modeled system. Therefore, to establish model sinks at each of the simulated fractures, drains were placed at several cells within the fracture portion of each fracture layer. The MODFLOW drain package is designed to simulate the effects of features such as agricultural drains, which remove water from the aquifer at a rate proportional to the difference between the head in the aquifer and some fixed head or elevation at the drain. The head computed by the model for the drain cell is actually an average value for the cell, and is normally assumed to prevail at some distance from the drain itself. The specified drain head prevails only within the drain and does not characterize the cell as a whole (McDonald and Harbaugh, 1988). Drains, therefore, constitute a "passive" sink, whereby water exits the model via gravity drainage.

For each drain cell, values for two parameters are required: the drain elevation and the conductance. For this model, the drain elevations were set at approximately 0.1 to 0.2 cm above their fracture layer bottom elevations. The second parameter is the conductance (CD) of the interface between the aquifer and the drain. This value represents an equivalent conductance describing all of the head loss between the drain and the region of the drain cell in which the average cell head can be assumed to prevail. The rate of drain water removal is thus proportional to the conductance. For the fracture well model, a conductance of 100 resulted in the numerical stability required to allow the model to converge.

6.3 Transient Simulations and Results

Three different fracture well scenarios were run to assess their dewatering effects after five and ten days of elapsed time:

- One eight-fracture well (seven fractures in the saturated zone)
- One three-fracture well (two fractures in the saturated zone)
- Combination of one eight-fracture well and one three-fracture well

The groundwater flow model transient simulations and results are summarized in Table 6.1. The predicted total cumulative water volume removed and the average extraction rate for each simulation are summarized in Table 6.2.

After each transient simulation, the extent of dewatering along the Row 29 cross-section was determined by examining the results for the uppermost layers and locating, for each column in Row 29, the top non-dry cell. In order to graphically display the results of a transient simulation, the modeled potentiometric heads for the top non-dry cells along Row 29 were tabulated and profiled.

6.3.1 One Eight-Fracture Well (Seven Fractures in Saturated Zone)

The five-day simulation predicted a maximum groundwater level drawdown of 102.3 cm (3.36 feet). The five-day horizontal radius of influence (measured as the horizontal distance from the well at which the minimum drawdown is 0.1 cm) was approximately 1,104 cm (36.2 feet). The ten-day simulation predicted a maximum groundwater level drawdown of 175.1 cm (5.74 feet). The ten-day modeled horizontal radius of influence was approximately 1,275 cm (41.8 feet). The predicted cross-section of the five- and ten-day dewatering effects for the eight-fracture well is shown in Figure 6.1.

The volumetric water balance for the model indicated that the modeled cumulative water volume removed from the model via all of the fracture drains was 11,273 gallons for the five-day simulation and 14,576 gallons for the ten-day simulation. Because the model represents only one quadrant of the total flow field surrounding the well, the total volume removed from the entire fracture well is calculated by quadrupling the volumes predicted by the model. The predicted total cumulative water volume removed from the eight-fracture well is 45,093 gallons over the first five days and 13,211 gallons over the second five days for a total of 58,304 gallons over ten days.

The average extraction rate is determined by dividing the predicted total cumulative four-quadrant volume removed by the elapsed time. The average extraction rate was 6.3 gpm over the first five days and 1.8 gpm for the second five days, for an average of 4.0 gpm over ten days. The five- and ten-day simulations represent early-time response during the expected total dewatering period. As more time elapses, the extraction rate (and drawdown) will continue to decrease toward a lower long-term extraction rate. As indicated above, a decline in the average extraction rate is already evident between five and ten days elapsed time. The extraction rate and drawdown will stabilize when the recharge within the zone of influence of the pumping well equals the rate of extraction from the well.

6.3.2 One Three-Fracture Well (Two Fractures in Saturated Zone)

The maximum groundwater level drawdown during the five-day simulation measured 17.1 cm (0.56 foot). The five-day horizontal radius of influence was approximately 954 cm (31.3 feet). The maximum groundwater level drawdown during the ten-day simulation measured 68.9 cm (2.26 feet). The ten-day horizontal radius of influence was approximately 1,275 cm (41.8 feet). The predicted cross-section of the five- and ten-day dewatering effects for the three-fracture well is shown in Figure 6.2.

The volumetric water balance for the model indicated that the modeled cumulative water volume removed from the model via all of the fracture drains was 4,303 gallons for the five-day simulation and 6,406 gallons for the ten-day simulation. The predicted total cumulative water volume removed from the three-fracture well is 17,211 gallons over the first five days and 8,413 gallons over the second five days for a total of 25,624 gallons over ten days. The average extraction rate was 2.4 gpm over the first five days and 1.2 gpm for the second five days, for an average of 1.8 gpm over ten days.

6.3.3 Combination of One Eight-Fracture Well and One Three-Fracture Well

This simulation was designed to replicate the first phase of the dewatering/SVE recovery test portion of the soil fracturing pilot test, where a vacuum would be simultaneously applied to one eight-fracture well and one three-fracture well, spaced 70 feet apart. For this simulation, the top non-dry cells' potentiometric head distribution results for the above-described eight-fracture and three-fracture well simulations were superimposed on a spreadsheet program to determine combined well interference effects. Because the two fracture wells are spaced 70 feet apart, only the results for the model cells within 70 feet (2,134 cm) from each well were included in the superposition. Using the MODFLOW results from the previous simulations, the drawdown effect of each well was reviewed, and the drawdowns were added to arrive at the cumulative drawdown due to well interference. The cumulative drawdowns were then used to calculate the estimated potentiometric head distribution for the two-well system.

The predicted cross-section of the five- and ten-day dewatering effects for the combination of the eight- and three-fracture wells, shown in Figure 6.3. At the five- and ten-day intervals, the combined well drawdown effects were minimal. Neither of the wells experienced an increase in the maximum drawdown due to well interference, and the maximum amount of drawdown superimposed in the interval between the wells was less than 1 cm. This can most likely be attributed to the relatively short time duration simulated in the model. The five- and ten-day simulation periods may not be of sufficient length to allow for the increase of drawdowns and radii of influence to a point where well interference effects may be more pronounced. However, since a small degree of combined well effects was observed, a longer time interval simulation would be expected to indicate an increase in the combined well drawdown effects.

Based on the results of the combined well interference analysis, the drawdowns calculated from the five- and ten-day pumping of the two-well system were not significantly different. Therefore, the cumulative removal volumes and extraction rates from the previous one-well simulations may be summed to provide an estimate of the combined effects of the two-well system. The predicted total cumulative water volume removed from both fracture wells is estimated to be 62,304 gallons over the first five days and 21,624 gallons over the second five days for a total of 83,928 gallons over ten days. The average extraction rate was 8.7 gpm over the first five days and 3.0 gpm for the second five days, for an average of 5.8 gpm over ten days.

7.0 PROJECT STATUS/ADDITIONAL MODELING ACTIVITIES

In their present configuration, the air and groundwater flow models are based on the pre-design assumptions that may not represent the actual fracture wells that will be used in the dewatering/SVE recovery test. Once the fracture well installations are complete and the unsaturated and saturated thicknesses, depth to bedrock and fracture spacings and depths are known, the following model updates and/or adjustments will be made:

- The horizontal dimensions of the groundwater model will be increased, and columns and rows will be added to place the constant-head boundaries further from the borehole corner;

- The vertical dimensions of both models will be adjusted to reflect the actual depths to the water table and bedrock measured in the field, and all elevations will be made relative to grade for ease of interpretation;

- The number of model layers and the fracture layer spacing in both models will be updated to better represent the actual spacing and estimated average propped thickness of the fractures observed in the fracture wells;

- The equivalent pneumatic or hydraulic parameters for the model layers containing fractures in both models will be updated to reflect the adjusted average fracture thickness, if necessary; and

- The areal extent of the modeled fractures in both models will be updated to better represent the areal extent of the propped fractures installed during the soil fracturing pilot test.

After the models are updated, the models will be used to provide predictive simulations for the pilot test early time response. After the data from the dewatering/SVE recovery portion of the soil fracturing pilot test have been reduced and analyzed, the model calibration will be carried out with the intent of adjusting the models' parameters so that the models simulate, as closely as possible, the pilot test early time response.

Field investigations have shown that hydraulic fractures typically exhibit some degree of fracture rise and asymmetry. Due to modeling limitations and complexities, the model will represent the equivalent vertical and horizontal fracture orientations and geometries. Therefore, the calibration procedure will focus on determining the set of values for aquifer parameters and stresses that provides the highest degree of reproduction of the field-measured flows, heads and volumes removed via extraction. During the model calibration, the soil fracturing pilot test pneumatic and potentiometric response data will be used to evaluate and modify, if necessary, the following parameters used in the air and groundwater flow models:

- Hydraulic conductivities and effective air permeabilities, which may include the equivalent values for the fracture zones; and

- Boundary conditions and the distances to the boundaries.

After using the soil fracturing pilot test data to calibrate the air and groundwater flow models, a fracture well configuration will be recommended based on information available to date. The recommended fracture well design will specify the number, depths and spacing of the fractures. A Conceptual Design Addendum will document the following:

- Model updates and adjustments;

- Recovery test response predictive simulations;

- dewatering/SVE recovery test data reduction and analysis;

- Model calibration and parameter adjustments; and

- Recovery test evaluation and recommended fracture well configuration.

After concurrence with the criteria identified in the Conceptual Design Addendum, the models will then be used to project long-term dewatering/SVE recovery well performance. The long-term performance projections will be used to aid in the design of a dewatering/SVE recovery wellfield and to determine if a full-scale dewatering/SVE remedial system using fracture wells is feasible. The following long-term performance projections will be obtained:

- The time required to dewater the overburden adjacent to a fracture well;

- The steady-state subsurface vacuum distribution after dewatering;

- The long-term air recovery rate from a fracture well after dewatering;

- The steady-state water table configuration adjacent to a fracture well after dewatering; and

- The long-term groundwater recovery rate from a fracture well.

The design of a dewatering/SVE recovery wellfield will consist of determining an optimal spacing, configuration and number of dewatering/SVE recovery wells for the effective dewatering and vapor recovery of the Zone 1 overburden.

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TABLE 4.1
MODEL INPUT HYDRAULIC AND PNEUMATIC PARAMETERS
FRACTURE WELL MODELING STATUS/METHODOLOGY
REMEDIATION OF ZONE 1
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
November 1995

MATERIAL OR LAYER		HYDRAULIC CONDUCTIVITY (ft/d)	HYDRAULIC CONDUCTIVITY (cm/s)	STORATIVITY	INTRINSIC PERMEABILITY (cm2)	EFFECTIVE AIR PERMEABILITY (cm2)	EFFECTIVE AIR POROSITY
UPPER TILL (0-20 FTBG)		0.01	3.53E-06	4.0E-03	3.60E-11	3.60E-12	0.1, 0.001
LOWER TILL (20-40 FTBG)		0.001	3.53E-07	4.0E-03	3.60E-12	NM	NM
SAND-PROPPED FRACTURE		150	5.29E-02	1.0E-02	5.39E-07	2.70E-07	0.3
COMPOSITE TILL/FRACTURE LAYER: EQUIVALENT VALUES							
- IN UPPER TILL:							
	HORIZONTAL	15.009	5.29E-03	NS	5.39E-08	2.70E-08	NS
	VERTICAL	0.011	3.92E-06	NS	3.99E-11	3.99E-12	NS
- IN LOWER TILL:							
	HORIZONTAL	15.001	5.29E-03	NS	5.39E-08	NM	NS/NM
	VERTICAL	0.001	3.92E-07	NS	3.99E-12	NM	NS/NM

NOTES: For fracture zones within fracture layers, equivalent horizontal and vertical hydraulic conductivities and effective air permeabilities were calculated using Equations (1) and (2) in Section 4.2.3.
Hydraulic conductivity (cm/s) = hydraulic conductivity (ft/d) x 3.527E-04
Intrinsic permeability (cm2) = hydraulic conductivity (cm/s) x 1.02E-05
Effective air permeability (cm2) = intrinsic permeability x relative permeability
Relative permeability = 0.1 for till, 0.5 for fractures (Section 2.3)
NS - Not specified - Equivalent storativities and effective air porosities were not specified for fracture layers.
NM - Not modeled - Air flow model did not simulate lower till.

TABLE 5.1
AIR FLOW MODEL - VOLUMETRIC FLOW RATES FOR STEADY-STATE SIMULATIONS
FRACTURE WELL MODELING STATUS/METHODOLOGY
REMEDIATION OF ZONE 1
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
November 1995

	-- VOLUMETRIC FLOW RATES (SCFM) --						
STEADY-STATE SIMULATION	FLOW OUT OF FIRST FRACTURE INTO WELL		FLOW OUT OF SECOND FRACTURE INTO WELL		TOTAL FLOW EXTRACTED FROM WELL	FLOW FROM ABOVE LAND SURFACE INTO OVERBURDEN	VOLUMETRIC BALANCE DISCREPANCY
EXAMPLE 01 - BOTH FRACTURES ON VACUUM - 0.6 ATMOSPHERE	0.052	+	0.036	=	0.088	0.088	0.000
EXAMPLE 02 - FIRST FRACTURE ON VACUUM - 0.6 ATMOSPHERE SECOND FRACTURE AT 1.0 ATMOSPHERE	0.124	+	-0.072	=	0.052	0.052	0.000

NOTES: Total Flow Extracted from Well = Flow out of First Fracture into Well + Flow out of Second Fracture into Well
Volumetric Balance Discrepancy = Flow from Above Land Surface into Overburden - Total Flow Extracted from Well

TABLE 6.1
GROUNDWATER FLOW MODEL - SUMMARY AND RESULTS OF SIMULATIONS
FRACTURE WELL MODELING STATUS/METHODOLOGY
REMEDIATION OF ZONE 1
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
November 1995

SIMULATED CONDITION	DESCRIPTION	MAXIMUM DRAWDOWN (cm)	MAXIMUM DRAWDOWN (ft)	HORIZONTAL RADIUS OF INFLUENCE (cm)	HORIZONTAL RADIUS OF INFLUENCE (ft)
I. One eight-fracture well (Seven fractures in saturated zone)	Seven fracture zones, each with several drains set at 0.1 to 0.2 cm above layer bottom elevation. Depths of drains, feet below grade: 14.23, 18.23, 22.23, 26.24, 30.24, 34.24, 38.24				
One eight-fracture well, 5 days	Length of stress period = 5 days (4.320E+05 seconds)	102.3	3.36	1,104	36.2
One eight-fracture well, 10 days	Length of stress period = 10 days (8.640E+05 seconds)	175.1	5.74	1,275	41.8
II. One three-fracture well (Two fractures in saturated zone)	Two fracture zones, each with several drains set at 0.1 to 0.2 cm above layer bottom elevation. Depths of drains, feet below grade: 18.23, 38.24				
One three-fracture well, 5 days	Length of stress period = 5 days (4.320E+05 seconds)	17.1	0.56	954	31.3
One three-fracture well, 10 days	Length of stress period = 10 days (8.640E+05 seconds)	68.9	2.26	1,275	41.8
III. Combination of one eight-fracture well and one three-fracture well	Superposition of results for one eight-fracture well and one three-fracture well, spaced 70 feet (2,134 cm) apart. Drain depths as indicated above. Drawdowns added to produce cumulative drawdowns due to well interference.				
Eight-fracture/three-fracture combination, 5 days	Length of stress period = 5 days (4.320E+05 seconds)	102.3*	3.36*	NA	NA
Eight-fracture/three-fracture combination, 10 days	Length of stress period = 10 days (8.640E+05 seconds)	175.1*	5.74*	NA	NA

NOTES: Horizontal Radius of Influence = the horizontal distance from the well at which the minimum drawdown is 0.1 cm
* - The Maximum Drawdowns for these simulations occurred adjacent to the eight-fracture well
NA - Result not applicable to this simulation.

TABLE 6.2
GROUNDWATER FLOW MODEL - PREDICTED TOTAL CUMULATIVE VOLUMES REMOVED AND AVERAGE EXTRACTION RATES
FRACTURE WELL MODELING STATUS/METHODOLOGY
REMEDIATION OF ZONE 1
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
November 1995

SIMULATED CONDITION	PREDICTED TOTAL CUMULATIVE VOLUME REMOVED (cm3)	(gal)	AVERAGE EXTRACTION RATE (cm3/s)	(gpm)
I. One eight-fracture well (Seven fractures in saturated zone)				
One eight-fracture well, days 1 - 5	170,676,000	45,093	395.1	6.3
One eight-fracture well, days 5 - 10	50,004,000	13,211	115.8	1.8
One eight-fracture well, days 1 - 10	220,680,000	58,304	255.4	4.0
II. One three-fracture well (Two fractures in saturated zone)				
One three-fracture well, days 1 - 5	65,144,000	17,211	150.8	2.4
One three-fracture well, days 5 - 10	31,844,000	8,413	73.7	1.2
One three-fracture well, days 1 - 10	96,988,000	25,624	112.3	1.8
III. Combination of one eight-fracture well and one three-fracture well				
Combination eight-fracture/three-fracture, days 1 - 5	235,820,000	62,304	545.9	8.7
Combination eight-fracture/three-fracture, days 5 - 10	81,848,000	21,624	189.5	3.0
Combination eight-fracture/three-fracture, days 1 - 10	317,668,000	83,928	367.7	5.8

NOTES: Predicted Total Cumulative Volume Removed (four quadrants) = Modeled Cumulative Volume Removed x 4
Average Extraction Rate = Predicted Total Cumulative Volume Removed/elapsed time

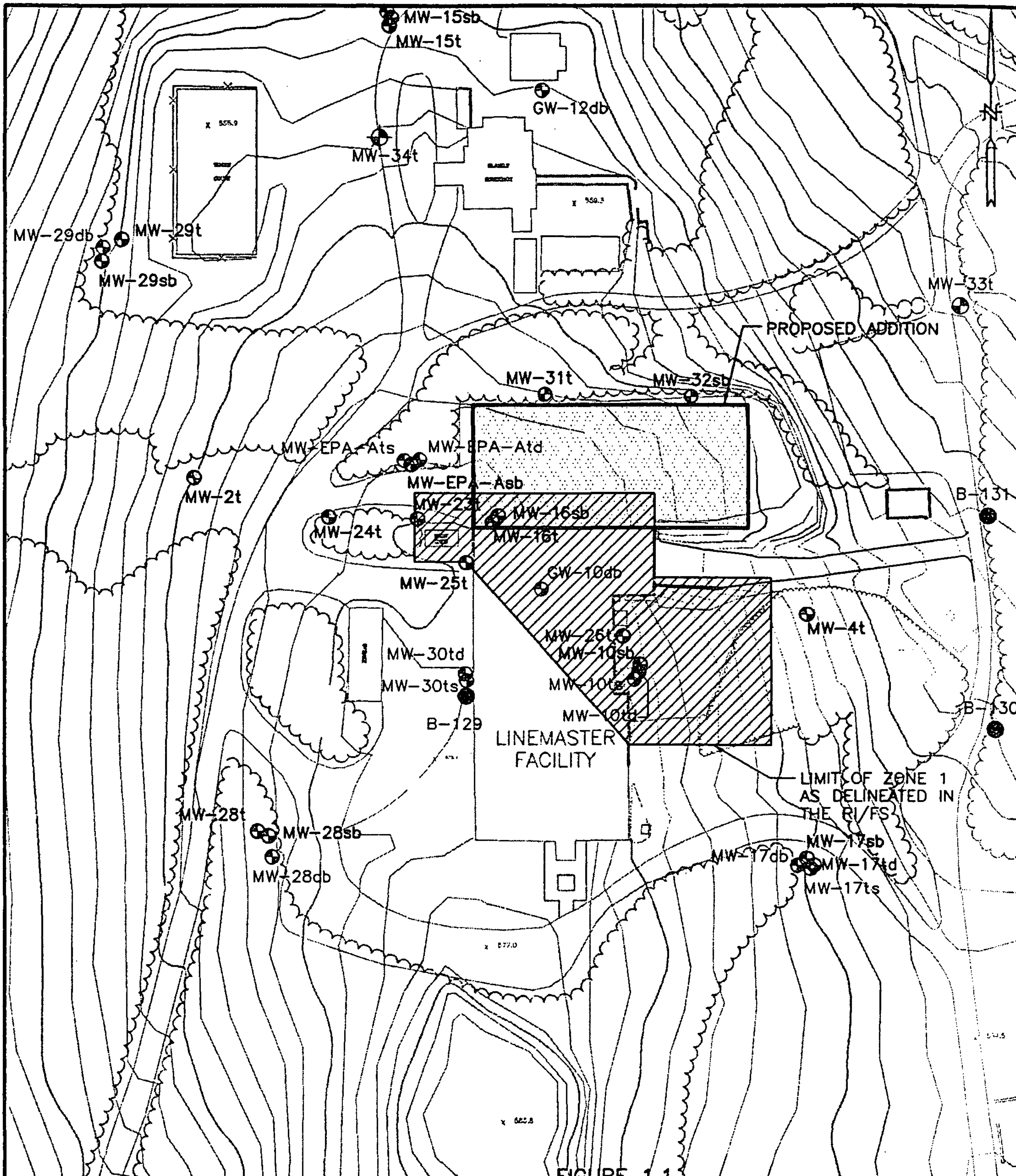

FIGURE 1.1

-100 0 100

SCALE: 1" = 100'

EXISTING BORING
B-130

EXISTING MONITORING WELL
MW-33t

FN: A5\86088PBW | PPP: ZONE1 | MS: S100 | UCS: WRLD

FUSS & O'NEILL INC. Consulting Engineers
146 HARTFORD ROAD, MANCHESTER, CONNECTICUT 06040
(203) 646-2469

ZONE 1 DELINEATION
FRACTURE WELL MODELING STATUS/ METHODOLOGY

REMEDIATION OF ZONE 1
LINEMASTER SWITCH CORP.

PLAINE HILL RD. WOODSTOCK, CT.

PROJ. NO.: 86-088A5 DATE: NOV. 1995 SCALE: 1"=100'

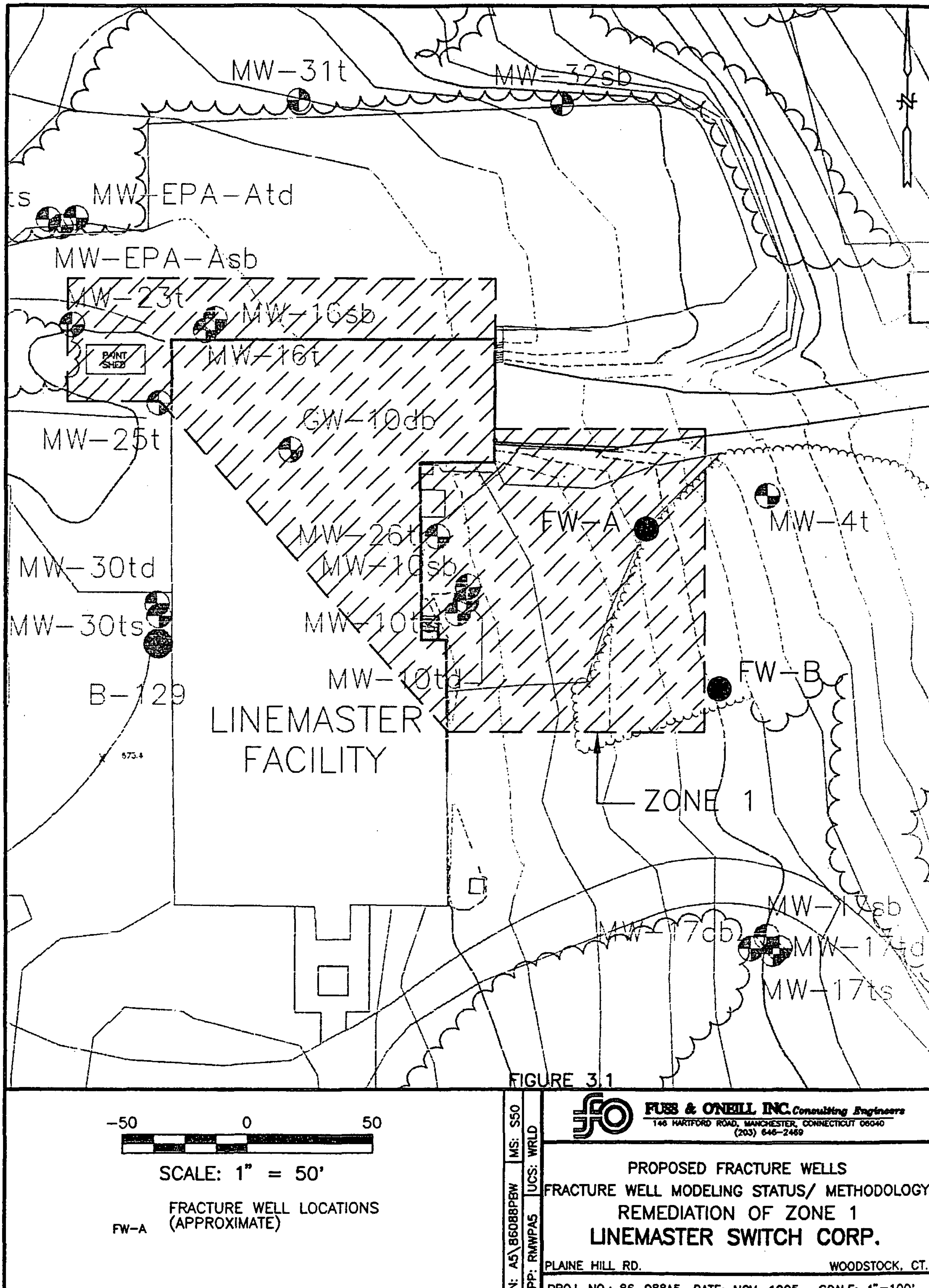

FIGURE 3.1

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146 HARTFORD ROAD, MANCHESTER, CONNECTICUT 06040
(203) 646-2469

PROPOSED FRACTURE WELLS
FRACTURE WELL MODELING STATUS/ METHODOLOGY
REMEDIATION OF ZONE 1
LINEMASTER SWITCH CORP.

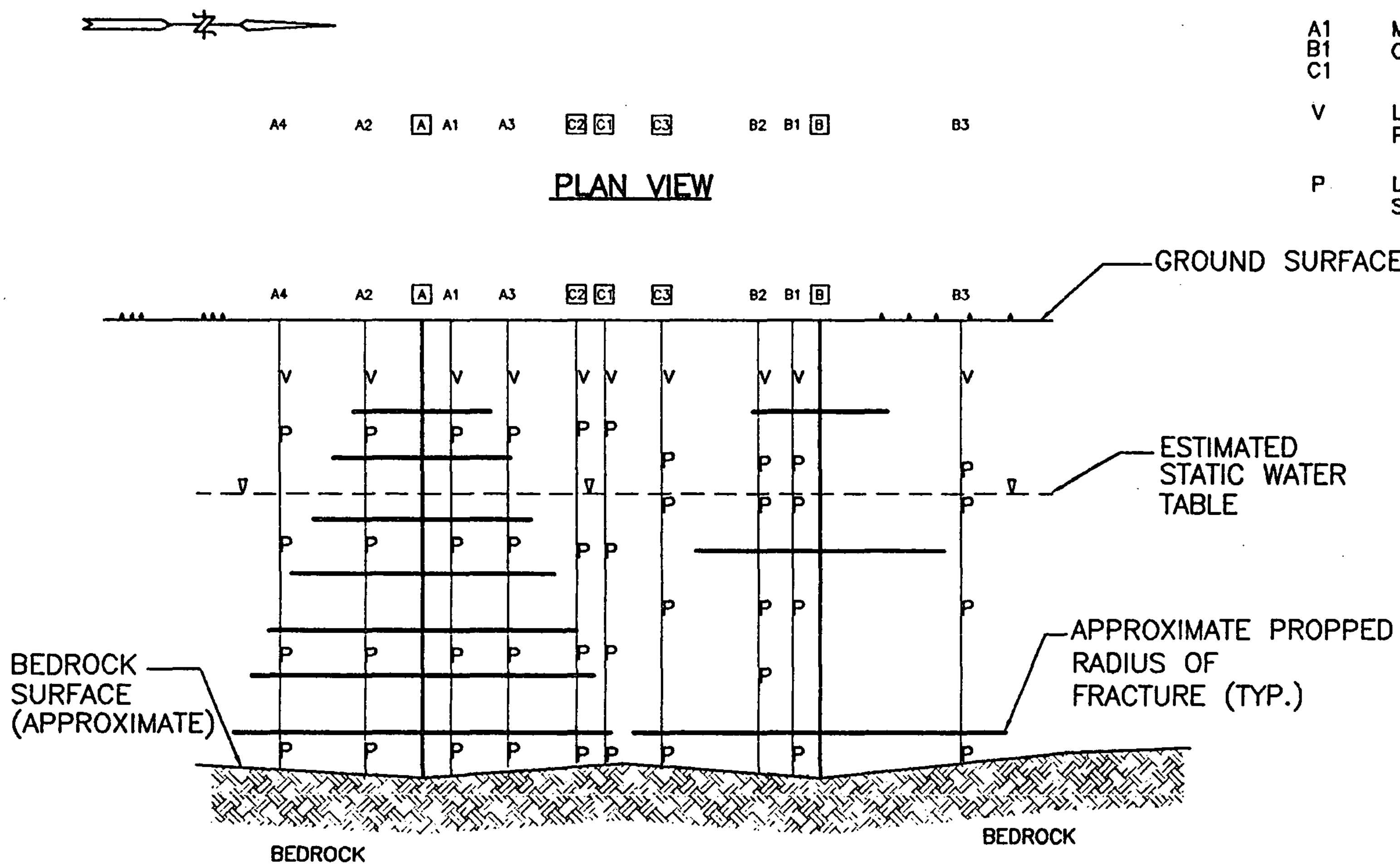
PLAINE HILL RD. WOODSTOCK, CT.

PROJ. NO.: 86-088A5 DATE: NOV. 1995 SCALE: 1"=100'

FN: A5\B6088PBW MS: S50
PPP: RMWPA5 UCS: WRLD

LEGEND

□	BORING INSTALLED BEFORE FRACTURING
A,B	FRACTURED WELLS
A1 B1 C1	MULTI–LEVEL PIEZOMETER CLUSTERS
V	LOCATION OF VAPOR PROBE (SHIELD POINT)
P	LOCATION OF PIEZOMETER SCREEN



CROSS–SECTION

NOTES:
1. ACTUAL FRACTURE SPACING AND PIEZOMETER LOCATIONS / DEPTHS WILL BE MODIFIED IN RESPONSE TO SITE CONDITIONS.
2. THE FRACTURES WILL LIKELY NOT REMAIN HORIZONTAL.

−30 0 30

SCALE: 1" = 30' (HORZ.)

−15 0 15

SCALE: 1" = 15' (VERT.)

FIGURE 3.2

FN: A5\86088OP1 MS: OPT1 PPP: UCS: WRLD

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CONCEPTUAL FRACTURE WELL AND PIEZOMETER CLUSTER LAYOUT
FRACTURE WELL MODELING STATUS/METHODOLOGY
REMEDIATION OF ZONE 1
LINEMASTER SWITCH CORP.
PLAINE HILL ROAD WOODSTOCK, CT.

PROJ. NO.: 86–088A5 DATE: NOV. 1995 SCALE: NOTED

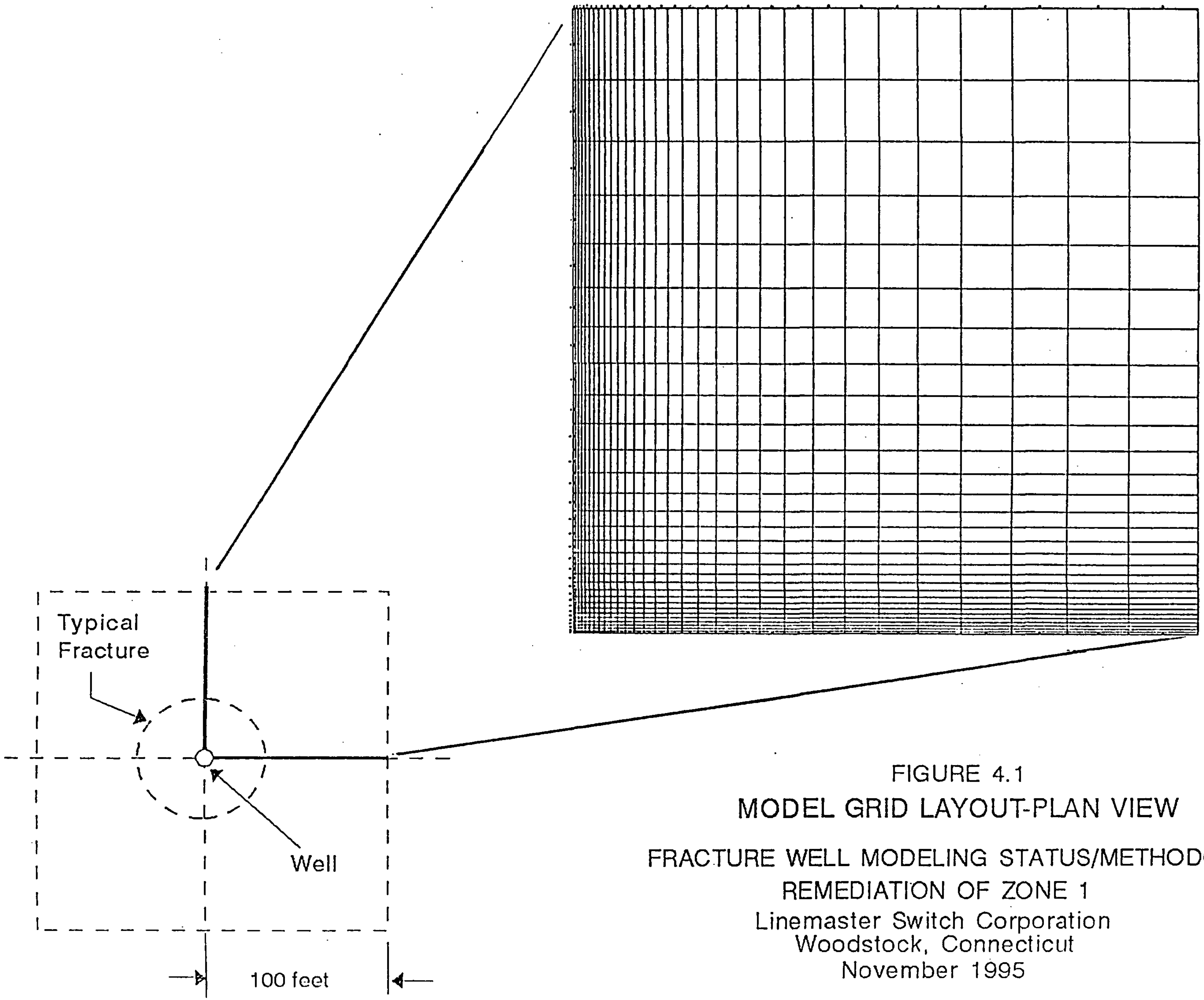

FIGURE 4.1
MODEL GRID LAYOUT-PLAN VIEW

FRACTURE WELL MODELING STATUS/METHODOLOGY
REMEDIATION OF ZONE 1
Linemaster Switch Corporation
Woodstock, Connecticut
November 1995

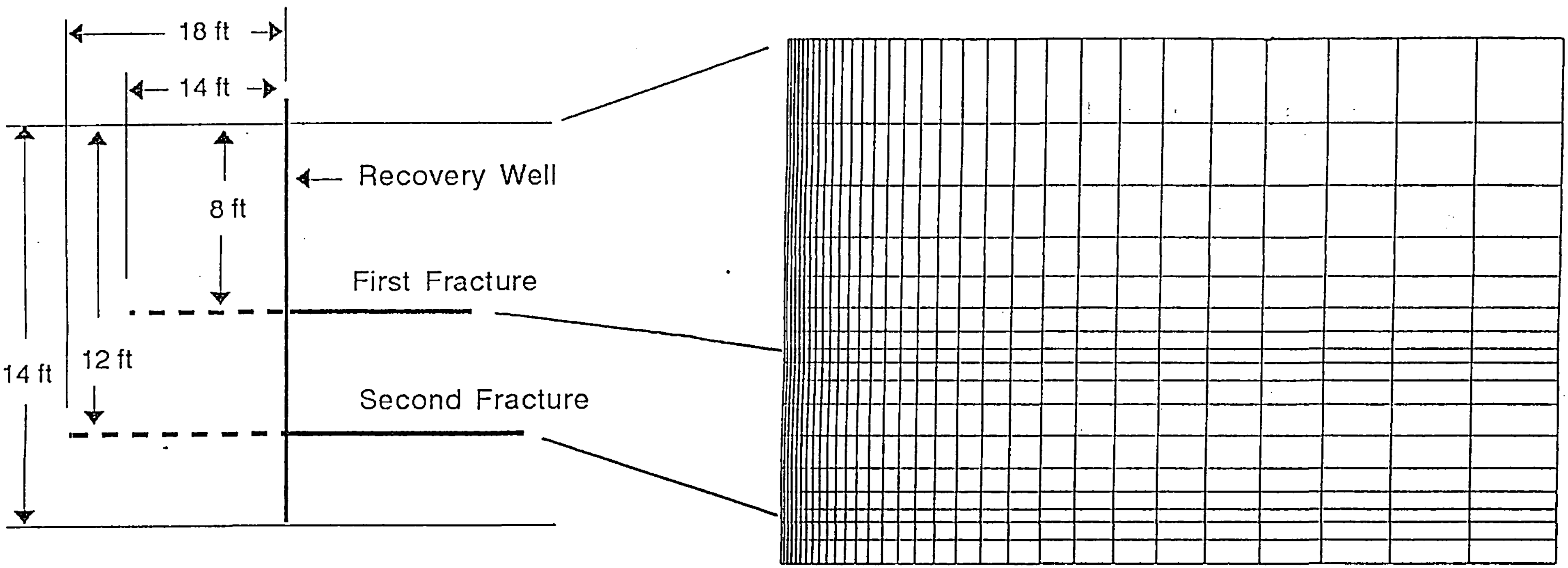

FIGURE 4.2

MODEL GRID LAYOUT

SECTION VIEW (AIR FLOW MODEL)

FRACTURE WELL MODELING STATUS/METHODOLOGY

REMEDIATION OF ZONE 1

Linemaster Switch Corporation

Woodstock, Connecticut

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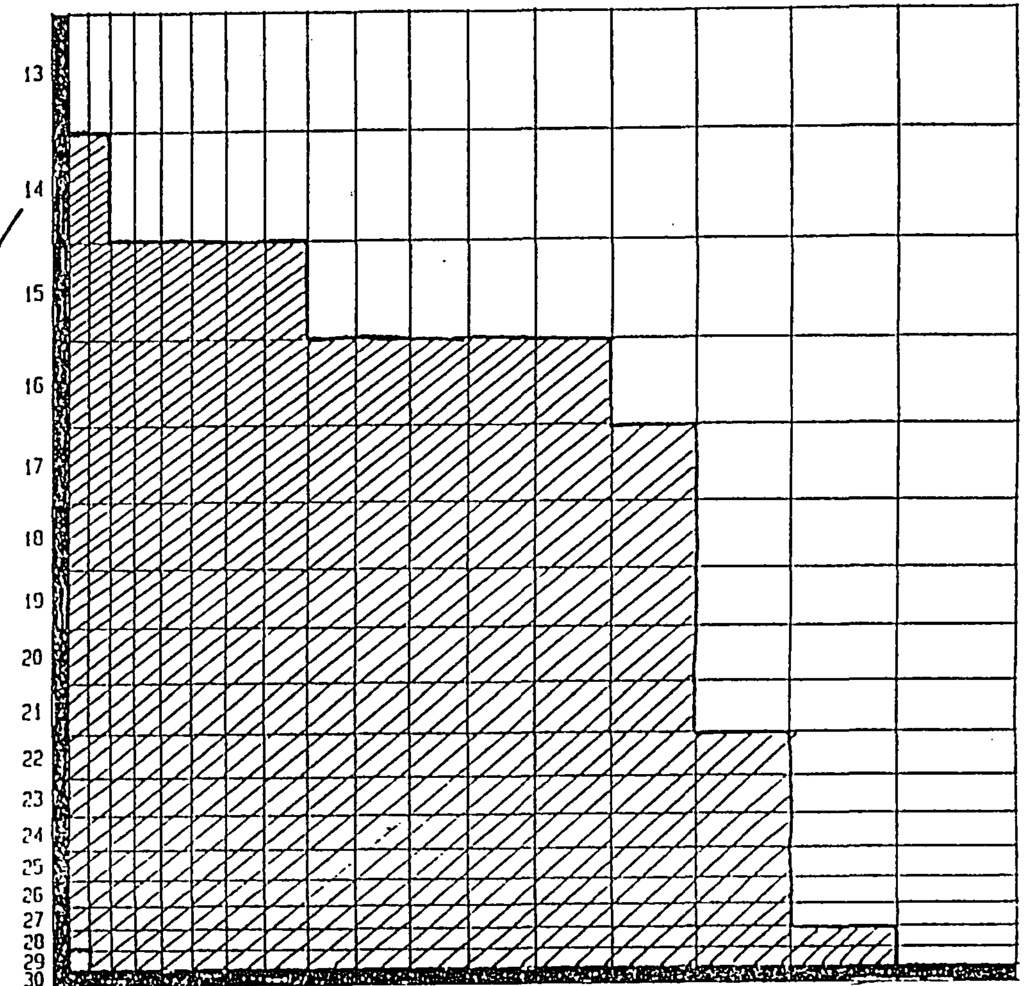
NOTES

1) Rows 1-12 Not Shown

2) Columns 19-30 Not Shown



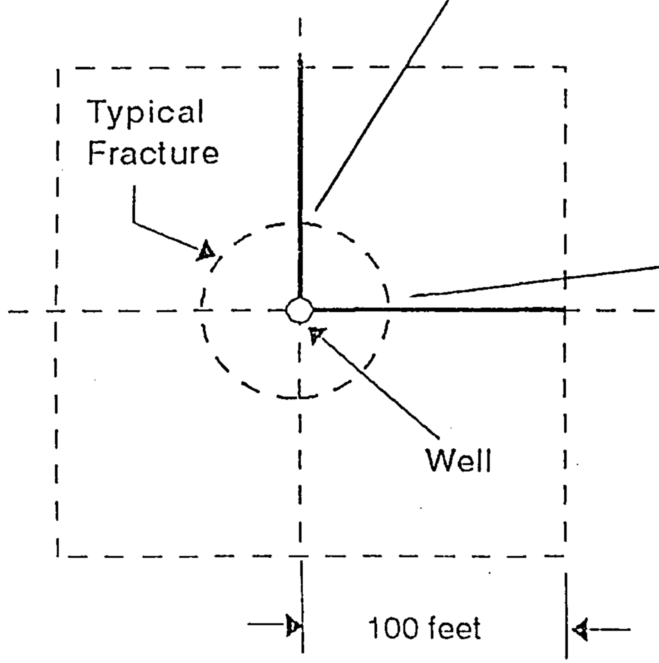


FIGURE 4.3

PLAN VIEW OF TYPICAL MODEL FRACTURE-CONTAINING LAYER

FRACTURE WELL MODELING STATUS/METHODOLOGY
REMEDIATION OF ZONE 1
Linemaster Switch Corporation
Woodstock, Connecticut
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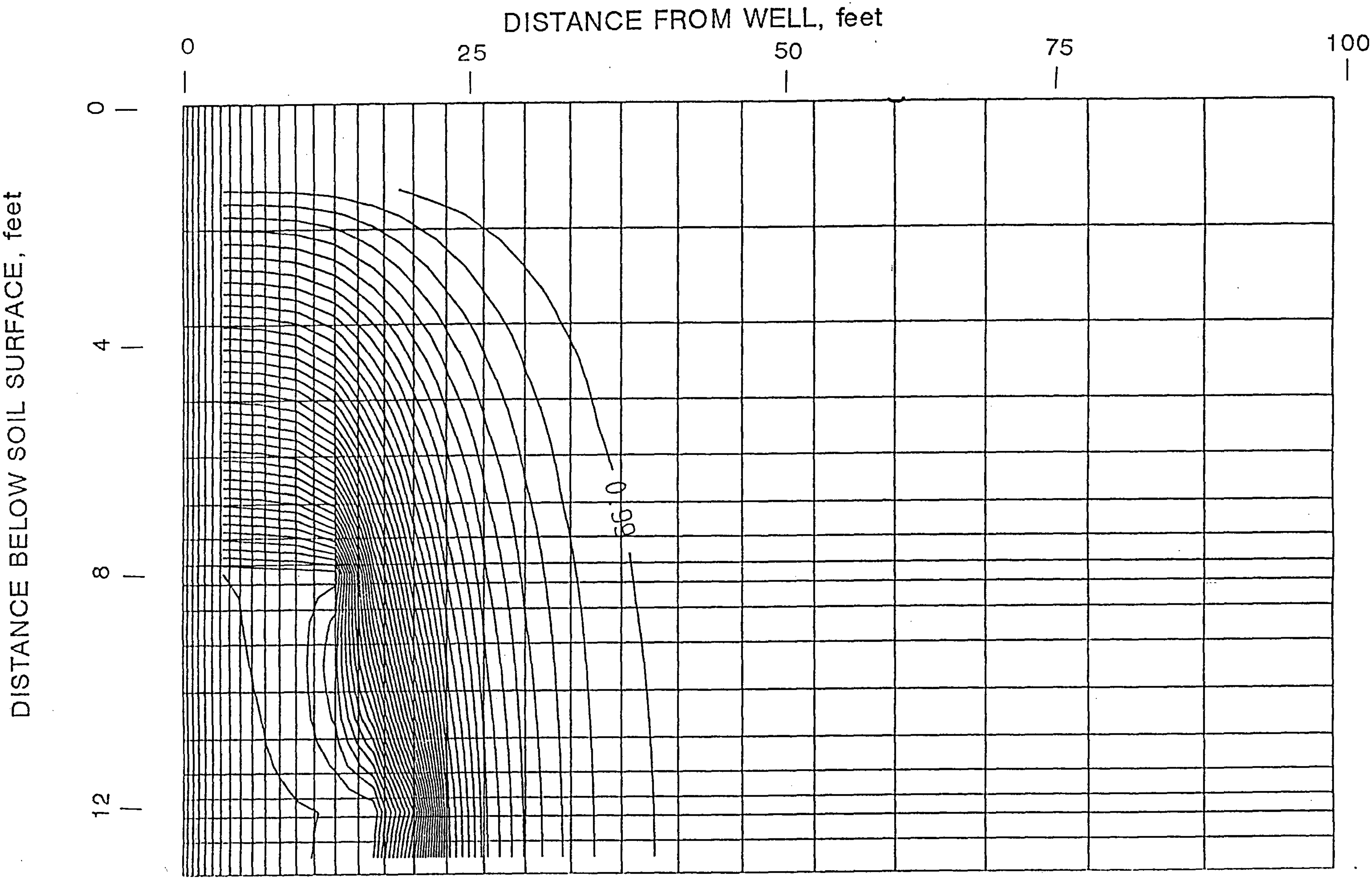

FIGURE 5.1
AIR FLOW MODEL
STEADY STATE RESULTS, FULL VACUUM
Pressure Range: 0.6-1.0 Atm
Contour Difference: 0.01 Atm

FRACTURE WELL MODELING STATUS/METHODOLOGY
REMEDIATION OF ZONE 1
Linemaster Switch Corporation
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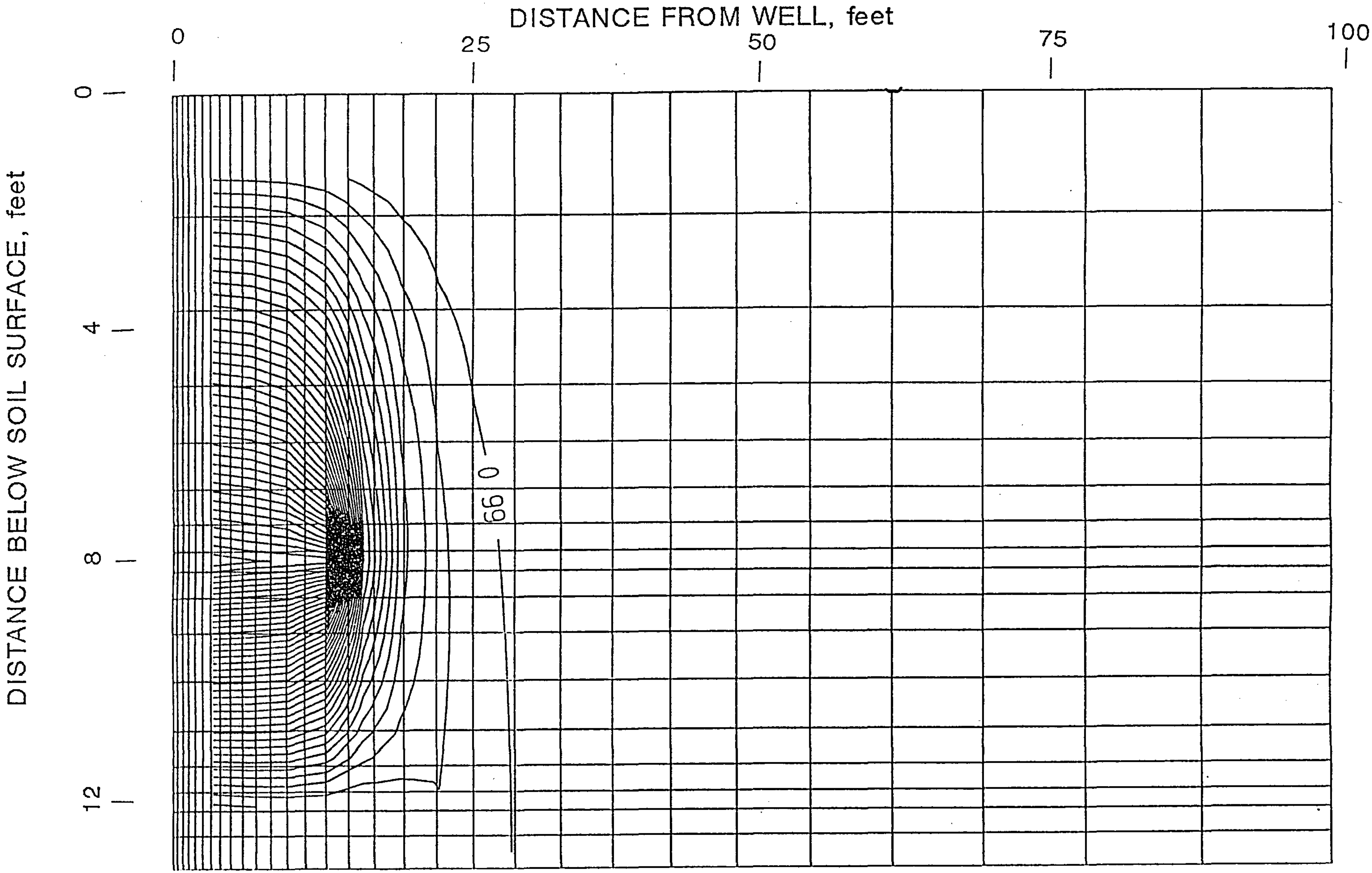

FIGURE 5.2
AIR FLOW MODEL
STEADY STATE RESULTS, ALTERNATING VACUUM
Pressure Range: 0.6-1.0 Atm
Contour Difference: 0.01 Atm

FRACTURE WELL MODELING STATUS/METHODOLOGY
REMEDIATION OF ZONE 1
Linemaster Switch Corporation
Woodstock, Connecticut
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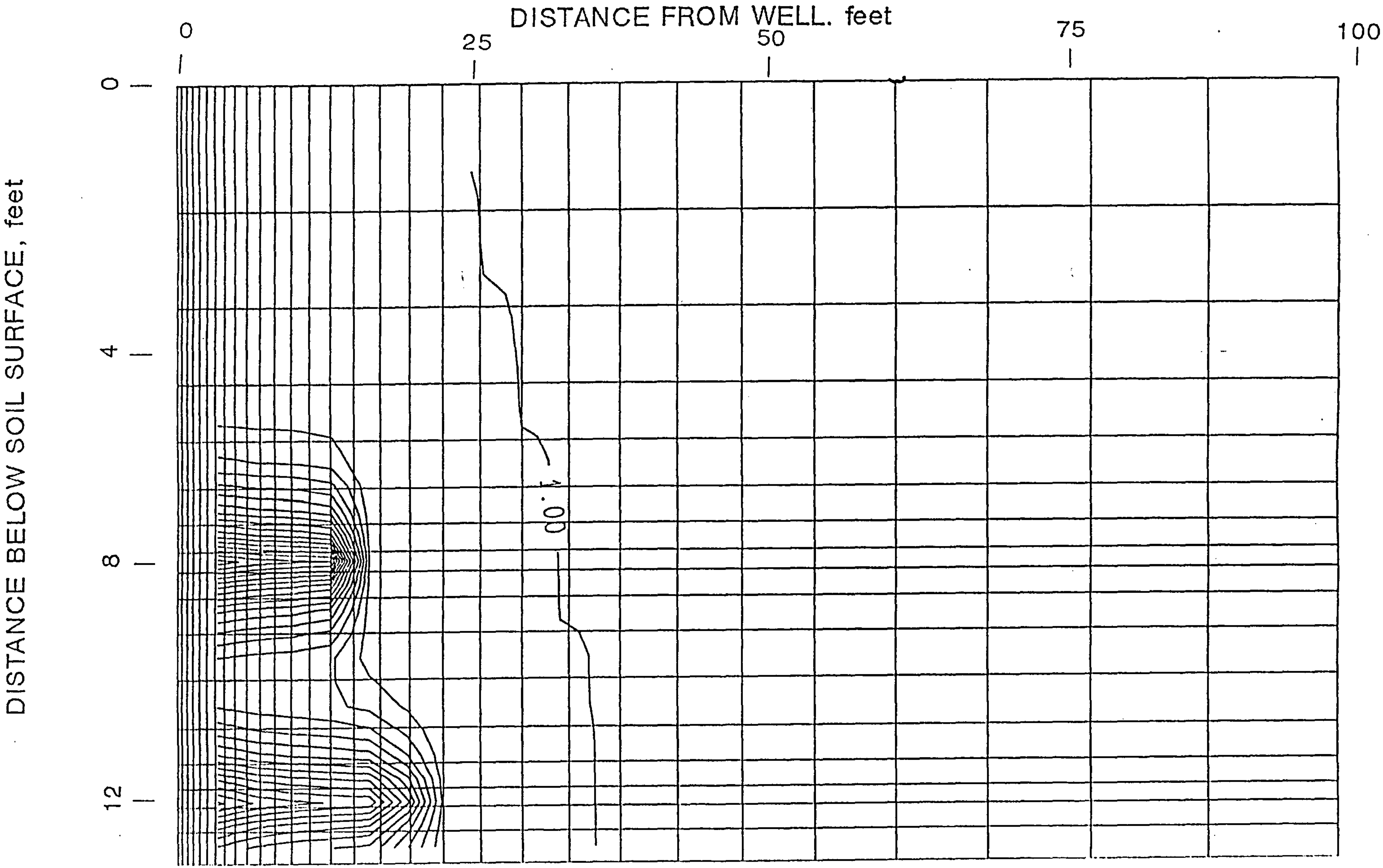

FIGURE 5.3

AIR FLOW MODEL

TRANSIENT RESULTS, FULL VACUUM AFTER ONE HOUR

Effective Air Porosity = 0.1

Pressure Range: 0.6-1.0 Atm

Contour Difference: 0.02 Atm

FRACTURE WELL MODELING STATUS/METHODOLOGY

REMEDIATION OF ZONE 1

Linemaster Switch Corporation

Woodstock, Connecticut

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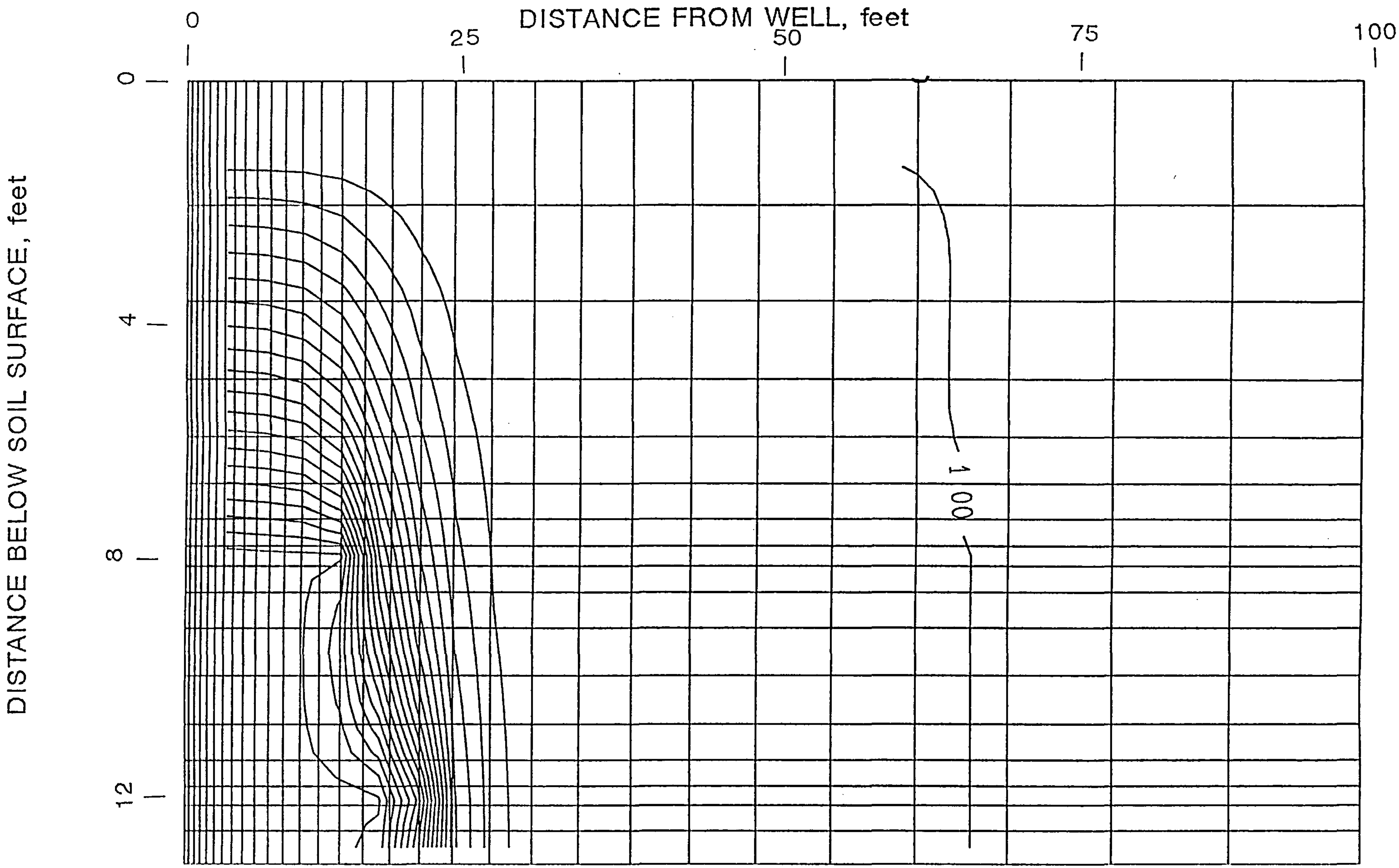

FIGURE 5.4

AIR FLOW MODEL

TRANSIENT RESULTS, FULL VACUUM AFTER 24 HOURS

Effective Air Porosity = 0.1

Pressure Range: 0.6-1.0 Atm

Contour Difference: 0.02 Atm

FRACTURE WELL MODELING STATUS/METHODOLOGY

REMEDIATION OF ZONE 1

Linemaster Switch Corporation

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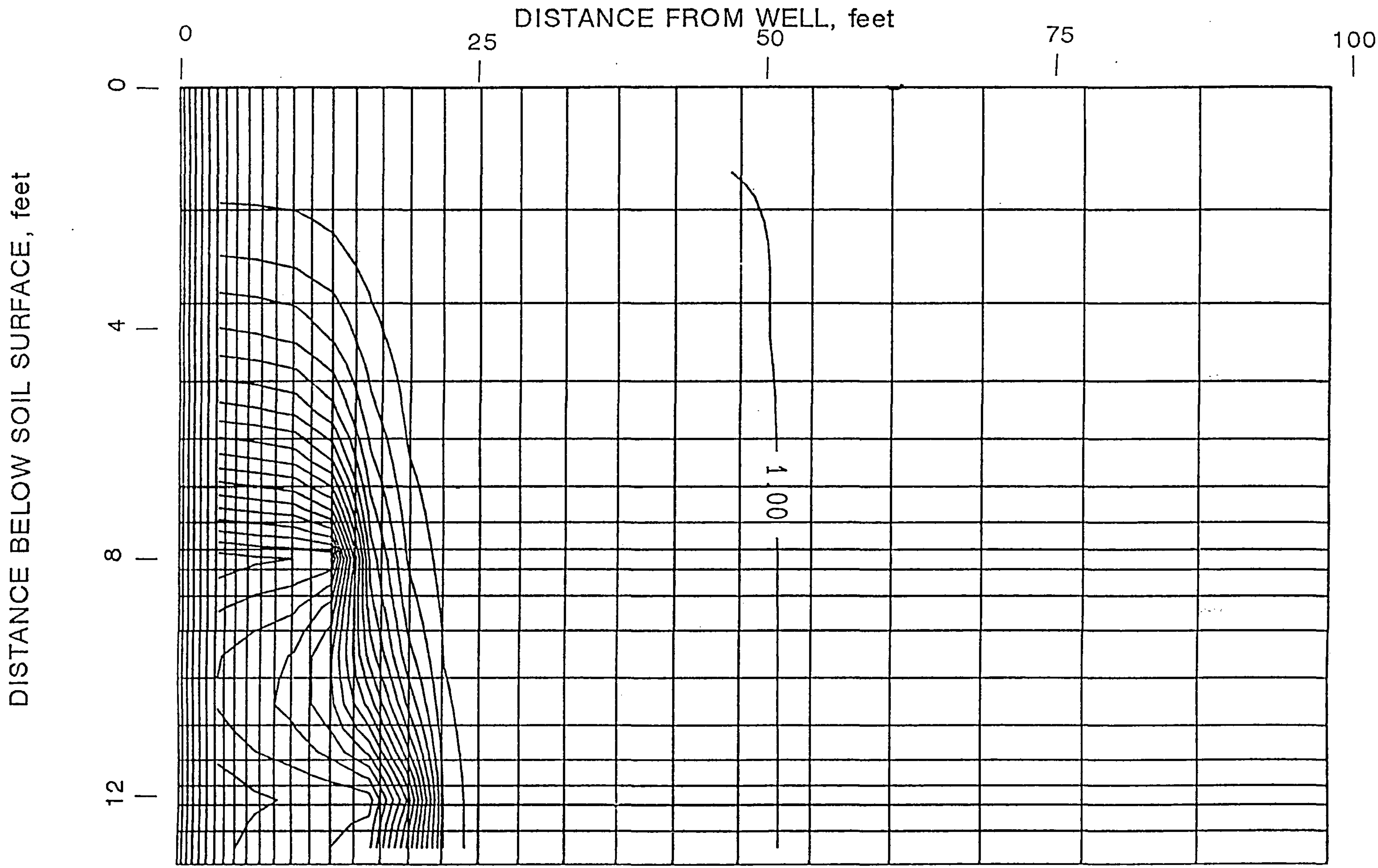

FIGURE 5.5

AIR FLOW MODEL

TRANSIENT RESULTS, FULL VACUUM AFTER ONE HOUR

Effective Air Porosity = 0.01

Pressure Range: 0.6-1.0 Atm

Contour Difference: 0.02 Atm

FRACTURE WELL MODELING STATUS/METHODOLOGY

REMEDIATION OF ZONE 1

Linemaster Switch Corporation

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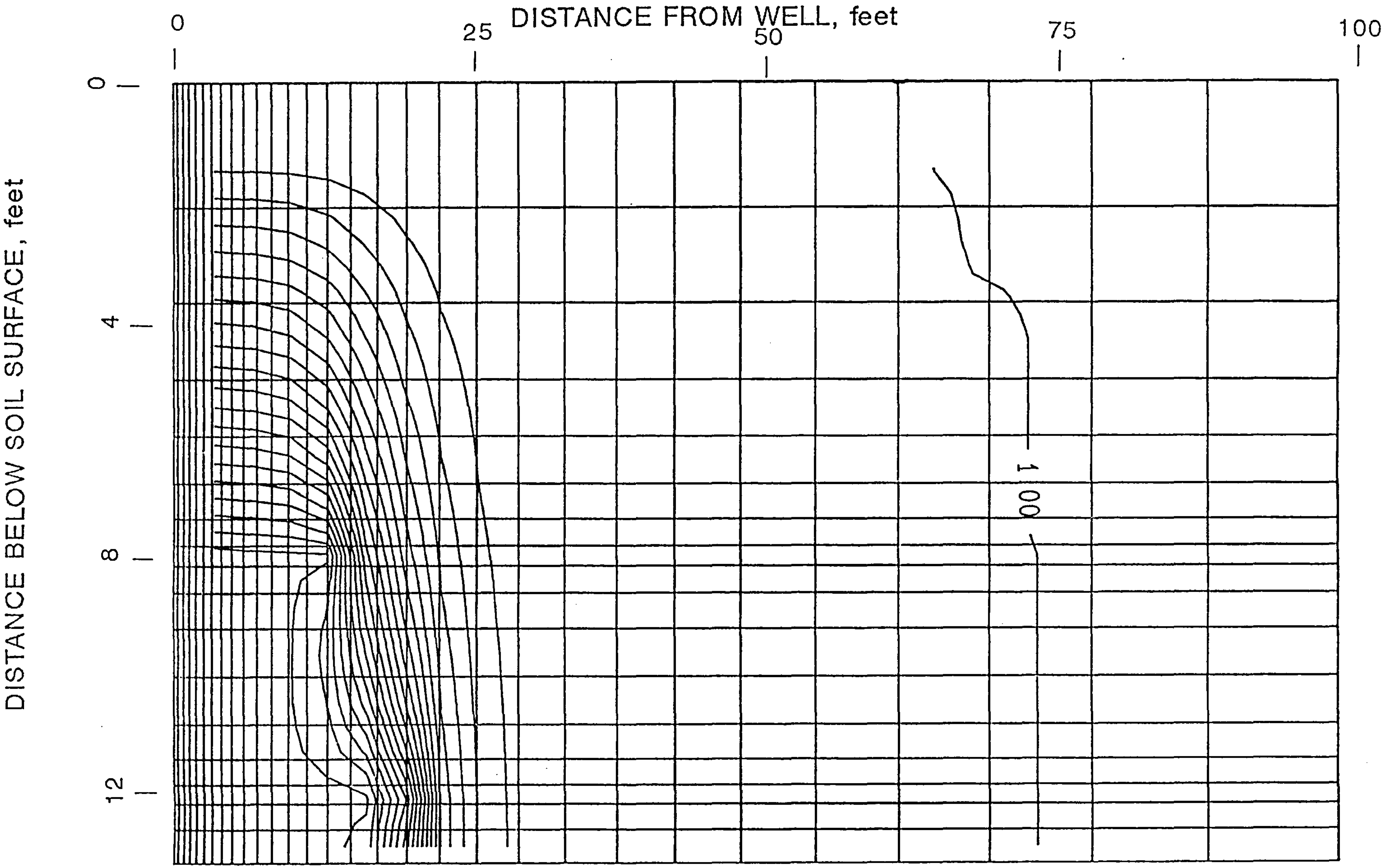

FIGURE 5.6

AIR FLOW MODEL

TRANSIENT RESULTS, FULL VACUUM AFTER 3.3 HOURS

Effective Air Porosity = 0.01

Pressure Range: 0.6-1.0 Atm

Contour Difference: [illegible] Atm

FRACTURE WELL MODELING STATUS/METHODOLOGY

REMEDIATION OF ZONE 1

Linemaster Switch Corporation

Woodstock, Connecticut

November 1995

FIGURE 6.1
GROUNDWATER FLOW MODEL - EIGHT-FRACTURE WELL TRANSIENT DEWATERING EFFECT
FRACTURE WELL MODELING STATUS/METHODOLOGY
REMEDIATION OF ZONE 1
LINEMASTER SWITCH CORPORATION
WOODSTOCK, CONNECTICUT
November 1995

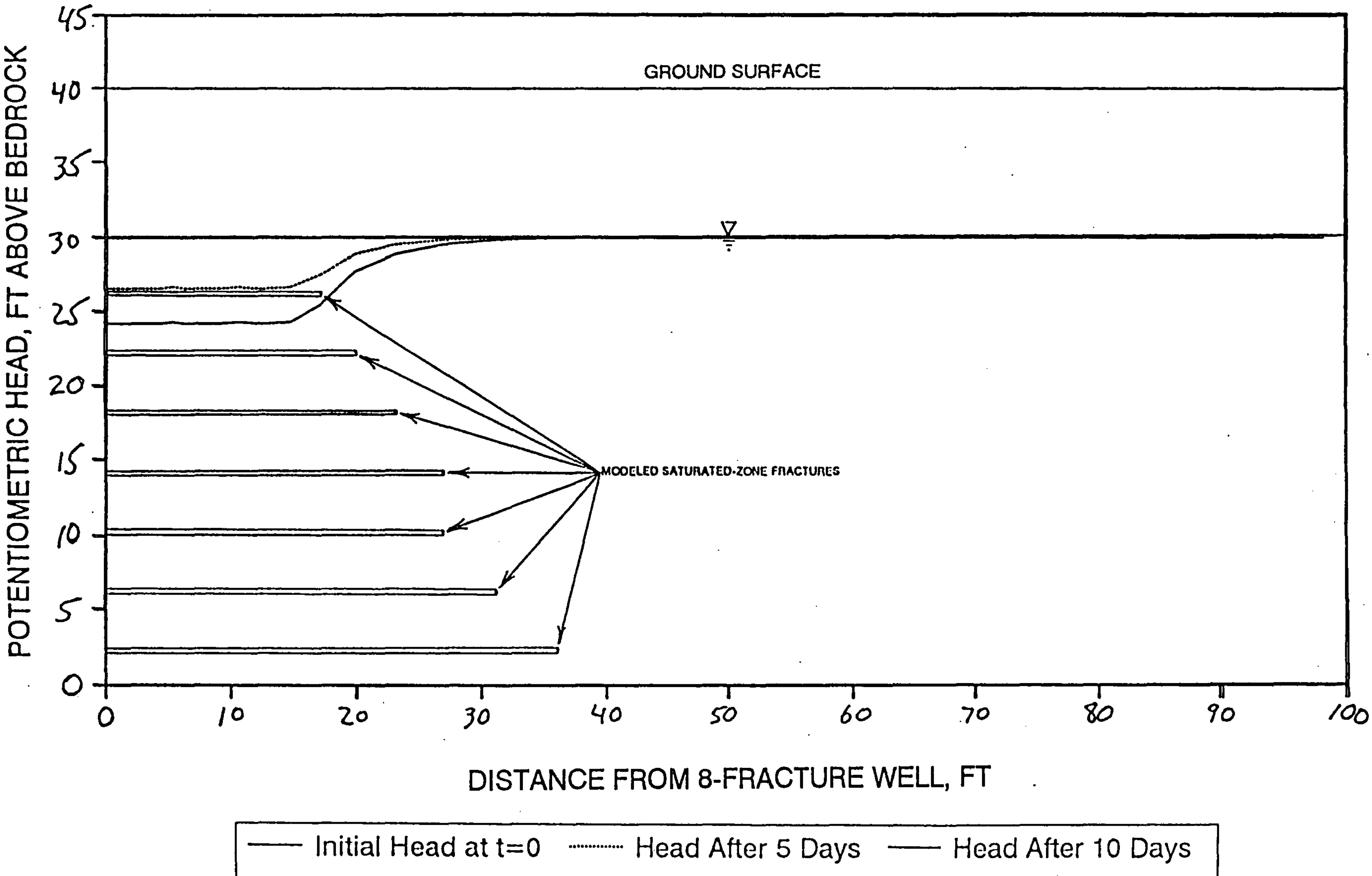

FIGURE 6.2
GROUNDWATER FLOW MODEL - THREE-FRACTURE WELL TRANSIENT DEWATERING EFFECT
FRACTURE WELL MODELING STATUS/METHODOLOGY
REMEDIATION OF ZONE 1
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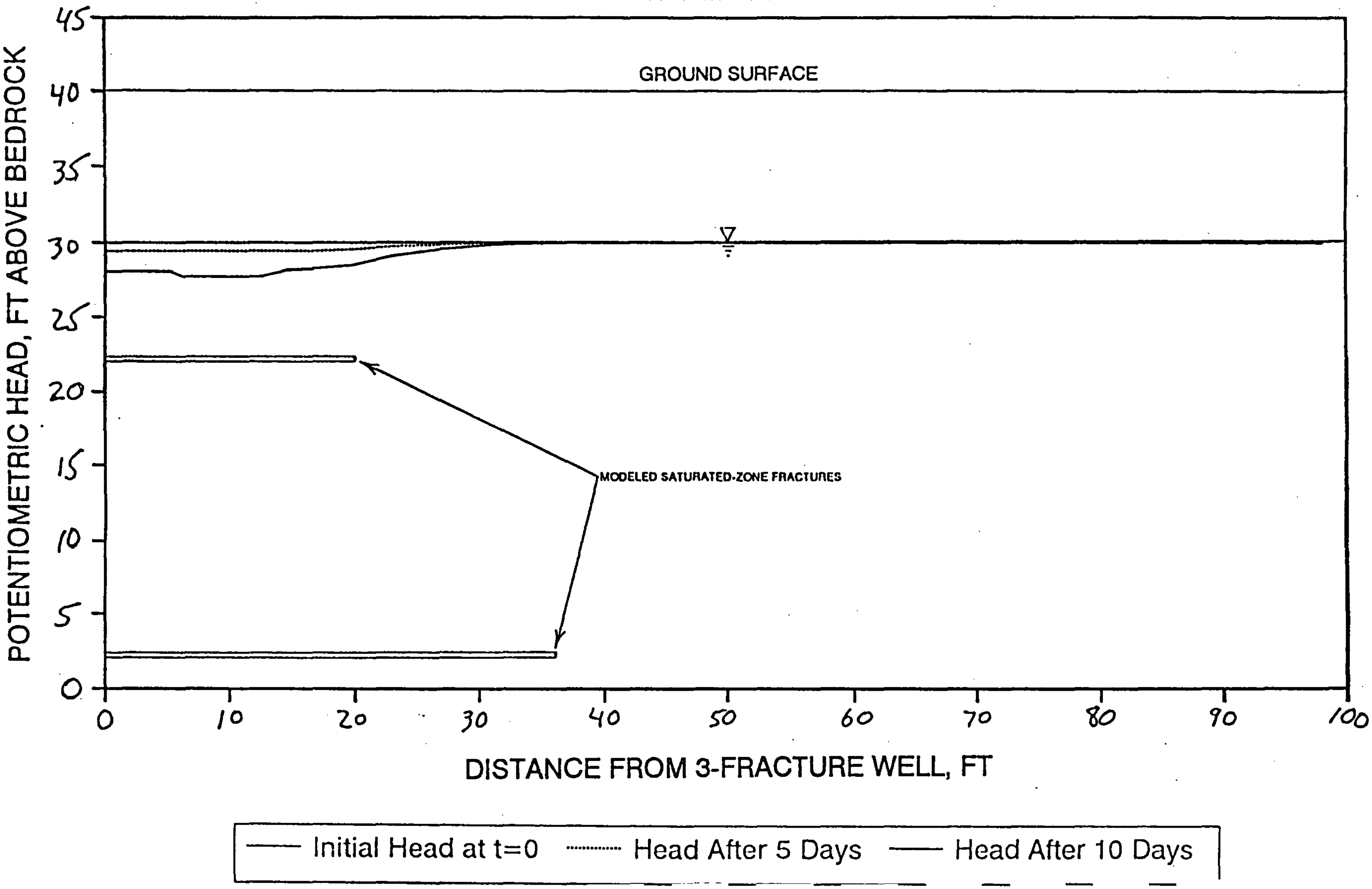

FIGURE 6.3

GROUNDWATER FLOW MODEL - EIGHT- AND THREE-FRACTURE WELL COMBINED TRANSIENT DEWATERING EFFECT

FRACTURE WELL MODELING STATUS/METHODOLOGY

REMEDIATION OF ZONE 1

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